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OF THE
39th ANNUAL CONVENTION

Of The
WESTERN CANADA WATER
and
WASTEWATER ASSOCIATION

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October 21, 22, 23 1987

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AUTOMATIC VALVES - THE AUTOMATIC ANSWER

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Western Regional Manager
Muesco Valve, Vancouver BC

Perhaps it is their appearance, with all that tubing, that makes automatic valves seem so complex and mysterious. As a result many who are involved with liquid handling simply do not understand the basic operating principles of these valves or their adaptability to solving a multitude of fluid handling problems. We will explain how these valves operate, cover basic application considerations and then briefly comment on their versatility.

A full description is 'automatic pilot-operated hydraulic control valves' which simply means:

(1) They are fundamentally self-regulating with no separate controller or control signal required.

(2) They use small externally mounted pilot devices to direct or 'pilot' the operation of the basic valve, much as a pilot controls a 747 jet.

(3) They are actuated or powered by the fluid handled. More specifically by differences in available fluid pressure - an important point.

(4) They can control a number of fluid parameters including pressure, flowrate, rate of change of flow or quantity of fluid passed. In all cases this is accomplished by control of valve position.

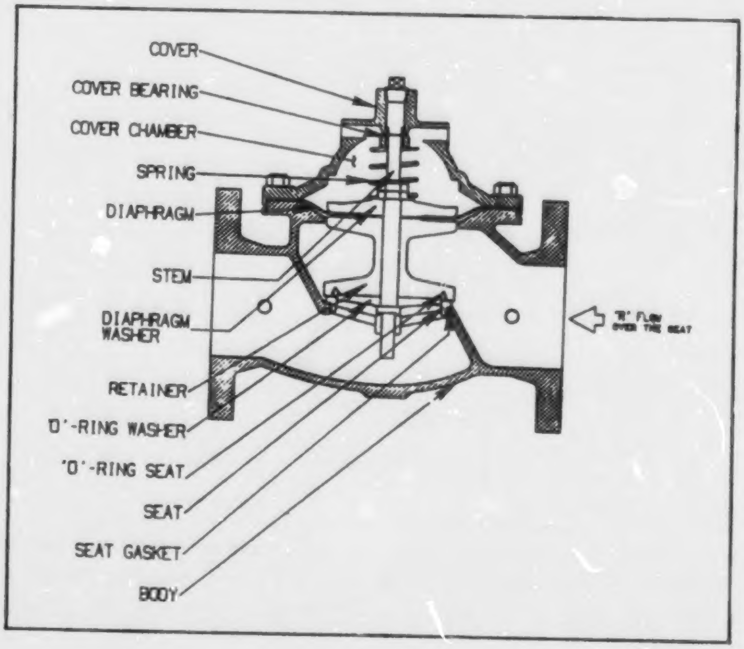

COVER
COVER BEARING
COVER CHAMBER
SPRING
DIAPHRAGM
STEM
DIAPHRAGM WASHER
RETAINER
'O'-RING WASHER
'O'-RING SEAT
SEAT
SEAT GASKET
BODY
'A' FLOW OVER THE SEAT

FIGURE 100

BASIC VALVE - GLOBE

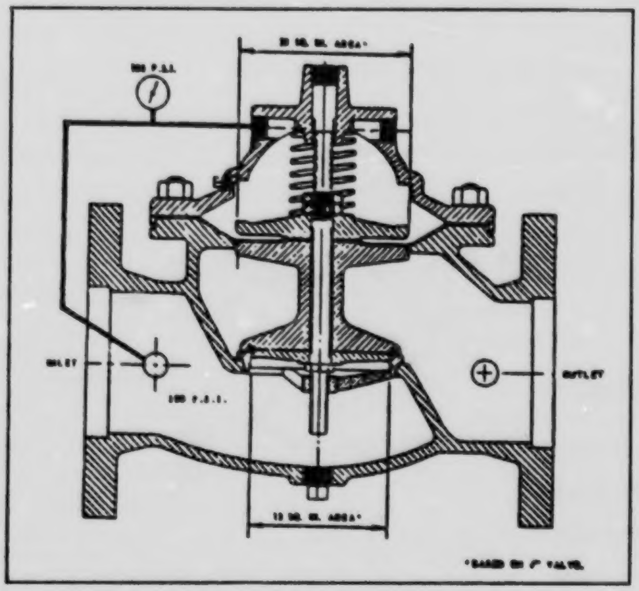
All automatic valves have a chamber with a sealed but movable actuator directly connected to a disc that can close onto a seat to control flow. This is of course a very generic description and while in some valves the chamber is a cylinder having a piston actuator and sealing rings, in others the actuator is a washer sealed to the chamber by a diaphragm. Some valves even use a flexible bladder that alone serves as the chamber, actuator and disc. For a number of very good reasons stem guided, diaphragm sealed valves are by far the most widely used variant.

Appreciate that while automatic valves are more often used to handle water they can also control other low viscosity fluids at moderate pressures and temperatures. Automatic valves are quite commonly used in aircraft fueling applications and the truck or railcar loading of fuels and liquified gases such as LPG and ammonia.

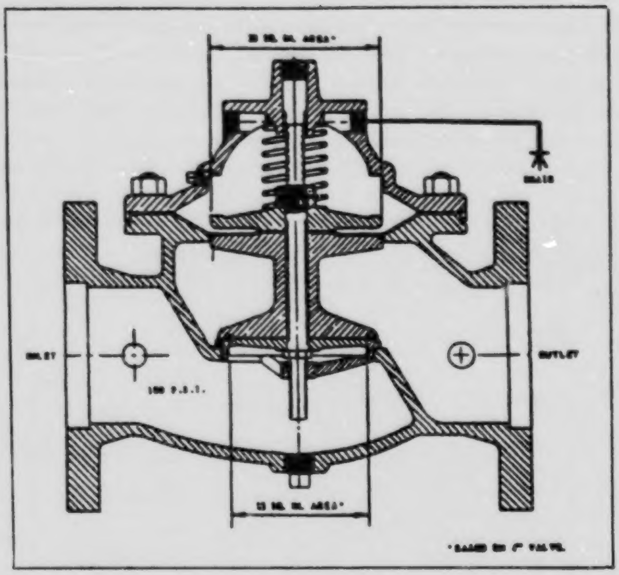
The basic operation of all automatic valves depends upon the fact that the actuator (diaphragm washer) area is significantly greater than the seat area and that fluid pressure differences are available. Our 4" valve, for example, has a diaphragm washer area of 20 sq.in. and a seat area of 13 sq.in. The ratio between these areas is called the 'trip' ratio and is about 1.5/1 with most valves. We can now learn how automatic valves work in three simple lessons.

LESSON 1

Let us assume that flow is from left-to-right and we have an upstream inlet pressure of 100 psig and a downstream outlet pressure of zero. If we run a tube from the inlet of our 4" valve up to the chamber we find there is an upward force on the bottom of the retainer (spool) of (area x psig) 13 x 100 = 1300 lb. but a downward force on the top of the retainer of 20 x 100 = 2000 lb. The result is a 2000 - 1300 = 700 lb. downward net force causing the valve to close (this downforce is also equal to area difference (20 - 13 =) 7 sq.in. x 100 psig). Thus with an inlet connection to the chamber the valve closes.



Now if we instead run a tube from the chamber to an open drain we have the same upward force of 1300 lb. but have eliminated the downward force (20 x 0 = 0) and the valve will open (the upforce is equal to the seat area of 13 sq.in. x 100 psig). Thus with an outlet connection to the chamber the valve opens.



Some valve pilots do exactly that, connect the chamber to inlet to close the main valve or connect the chamber to drain to open it. Our Figure 10-14 float valve is a good example. When the reservoir level is full the pilot connects the chamber to inlet so the main valve closes. But when the reservoir level falls to a preset level the pilot connects the chamber to drain and the main valve opens.

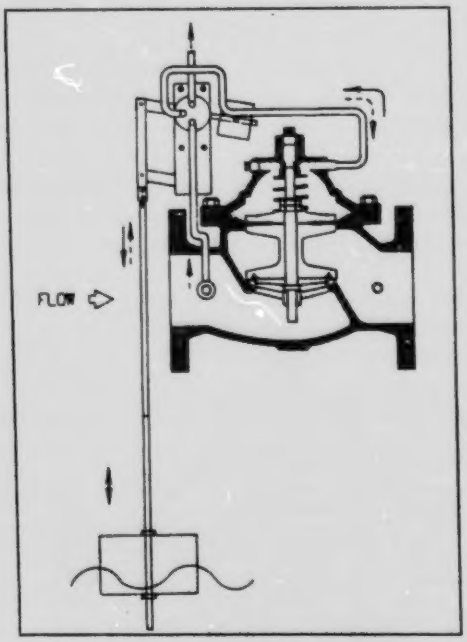
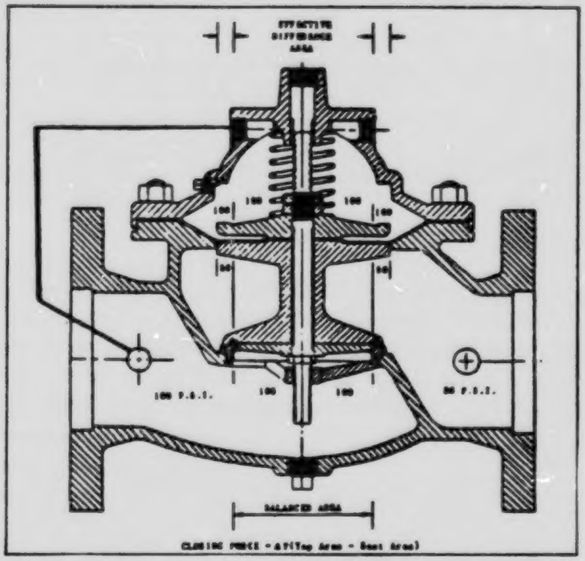


FIGURE 110-14

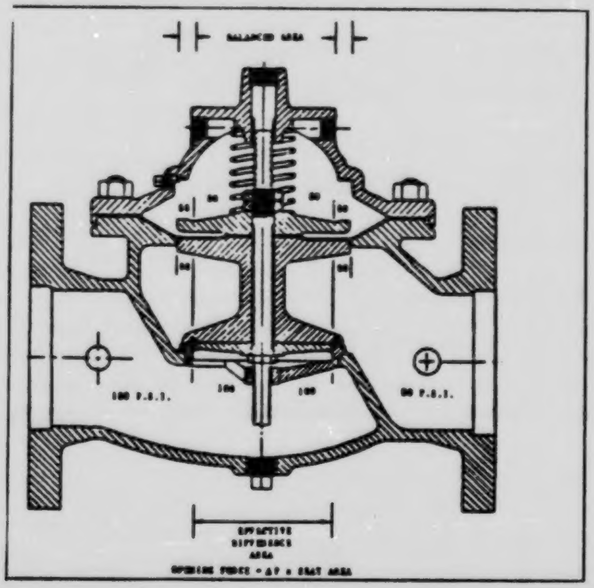
This is known as a DIRECT ACTING (or non-modulating) type of valve as it is either open or closed.

LESSON 2

Let us now assume the downstream outlet pressure is 80 psig and see what happens when we run a tube from the inlet to the chamber. The easiest way to understand the forces involved is to appreciate that the area at the top of the retainer equal to the seat area will produce a downforce equal to the upforce on the bottom of the retainer. Only that area at the top of the retainer **exceeding** the seat area is actually effective and the difference between valve inlet pressure (above) and outlet pressure (below) acting on that area gives us a net downforce. In our example the difference in areas is 20 − 13 = 7 sq.in. and the difference in pressures is 100 − 80 = 20 psig. Thus the net downforce created is 7 x 20 = 140 lb. and the valve closes. As before an **inlet connection** to the chamber and the **valve closes**.


EFFECTIVE DIFFERENCE AREA
100
100
100
100
80
80
100 P.S.I.
100
100
80 P.S.I.
BALANCED AREA
CLOSING FORCE = ΔP(Top Area − Seat Area)

When we then connect the chamber to the valve outlet with a tube the area at the top of the retainer exceeding the seat area now has the same pressure on both sides. However if we look at the area **equal** to the seat area we find that the difference between valve outlet pressure (above) and inlet pressure (below) acts on that area to create a net upforce. In our example the seat area is 13 sq.in. and the difference in pressures is 100 − 80 = 20 psig. Thus the net upforce created is 13 x 20 = 260 lb. and the valve opens. Again an **outlet connection** to the chamber and the **valve opens**.


BALANCED AREA
80
80
80
80
80
80
100 P.S.I.
100
100
80 P.S.I.
EFFECTIVE DIFFERENCE AREA
OPENING FORCE = ΔP x SEAT AREA

Our Figure 127 altitude valve effectively operates in this manner. The pilot will connect the chamber to inlet to close the valve when the reservoir is full and connect the chamber to outlet to open the valve when the reservoir is empty.

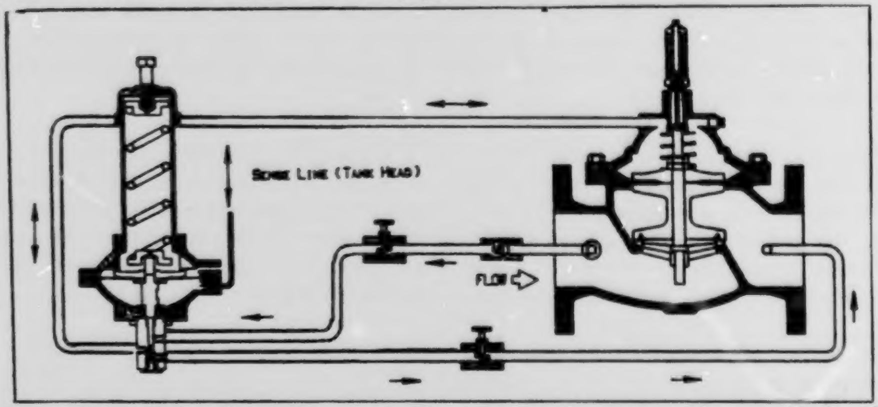


FIGURE 127

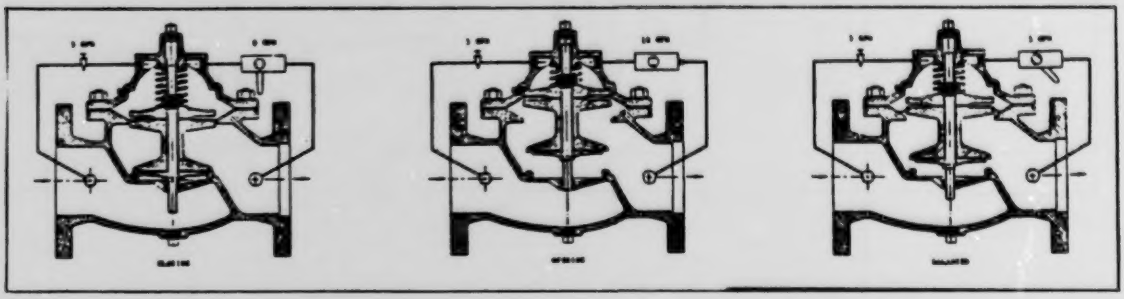
Notice that while the valve behaves in the same basic manner the magnitude of the forces generated will decrease in proportion to the decrease in pressure difference between inlet and outlet. Nearly all automatic valves are powered by the difference between inlet and outlet pressures and if those pressures should ever equalize the valve has no closing force at all, same for the weight of the moving assembly [assuming the stem is vertical]. Thus condition can occur if the automatic valve is even partially open and a downstream manual valve is suddenly closed.

The spring commonly provided atop the diaphragm washer is primarily intended to ensure there is a minimum pressure drop across the valve before it can open. This is turn assures a reasonable closing force can be generated when needed. Should inlet and outlet pressures ever equalize the spring will also tend to speed up the slow return of the valve to a closed position so that a pressure difference can again be established.

LESSON 3

Many variations of automatic valve are classified as MODULATING. This is just a fancy word for regulating or adjusting to suit conditions. Instead of being either full open or full closed the valve is usually at some intermediate position to control the desired fluid parameter.

Let us use the previous example but connect **both** the inlet and the outlet to the valve chamber. Next we will place a restriction in the line leading from the inlet to the chamber. This can be an orifice, needle valve or nozzle and while flow through this restriction will vary with pressure differences let us assume it is 5 GPM. Now we then put a manual pilot valve in the line from the chamber to the outlet and assume it can pass 10 GPM when full open.



When we close our manual pilot valve we basically have a connection (albeit restricted) between the inlet and the chamber and the main valve closes. If we then fully open our manual pilot valve we have a 5 GPM flow into the chamber - but a 10 GPM flow out. Since we are allowing more fluid out of the chamber than we can move in we effectively have connected the chamber to outlet and the valve will open. If we throttle our manual pilot valve until it just passes 5 GPM the main valve will remain in whatever position it was at since flow into the chamber is balanced by flow out and the valve cannot move.

Effectively the main valve simply amplifies the actions of the pilot device. All of our modulating pilots are by themselves small direct acting devices capable of controlling fluid at low flows. These include the Figure 10-11 modulating float, Figure 14 rate of flow, Figure 15 pressure reducing and Figure 16 pressure relief pilots. When installed between the chamber and outlet of a large valve having the inlet to chamber line restriction necessary for modulating action they 'pilot' the valve causing it to duplicate their function.

By far the majority of automatic valves are comparable to our example. However there are several interesting main valve variations available.

REVERSE FLOW

Most waterworks valves have flow directed first under the seat and then leaving over the seat. This 'under/over' flow is indicated by a symbol representing the valve seat and internal webbing plus a flow direction arrow. Interestingly the valve will also function if it is reversed so that flow goes 'over/under'. In fact our Figure 118R check valve is simply a basic valve with one tube to the chamber. Flowing one way this is an outlet connection and the valve opens, while flowing the other way it is an inlet connection and the valve closes!

FIGURE 100 BASIC VALVE

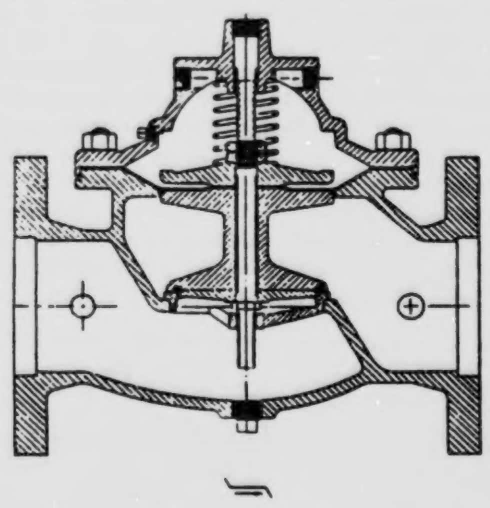
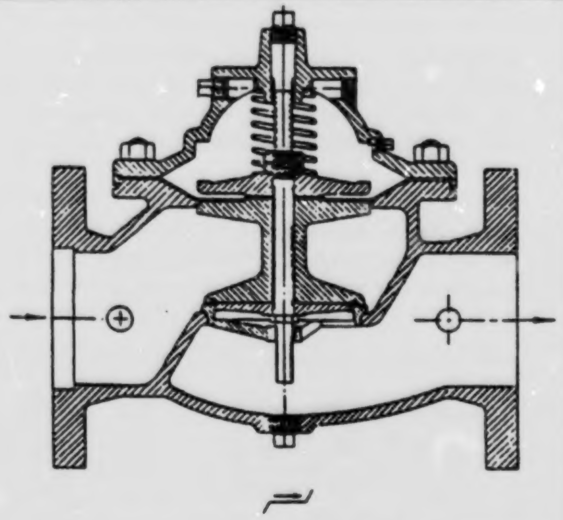
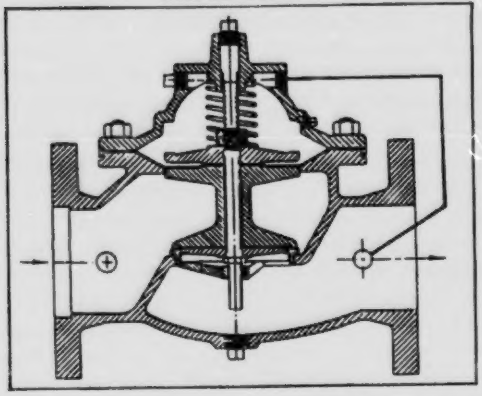


FIGURE 100R BASIC VALVE



CHECK VALVE - DIAPHRAGM TYPE
FIGURE 118R



While there are other factors involved the main consideration in selecting flow direction is what happens should the diaphragm ever fail. With the modern nylon reinforced synthetic diaphragms used today a rupture failure is quite uncommon however should one occur with an 'under/over' valve there is effectively a large chamber-to-outlet connection and the valve fails open. With reverse 'over/under' flow such a failure causes a large chamber-to-inlet connection and the valve fails closed. For obvious reasons reverse flow is virtually standard when handling petroleum products. It is also used occasionally in irrigation service when a valve failing open could flood out a field.

DUAL CHAMBER

Since single chamber valves are in fact powered by the difference between inlet and outlet pressures there must always be some pressure loss across the valve to ensure a reasonable closing response. In some applications, such as in-line pump control valves, this pressure loss may not be desirable. Further, if the pressure difference rises significantly as the valve closes due to the rise of the pump curve and drop in line friction our valve closing rate will accelerate. Problems of pressure loss and initial slow response coupled with final slow closure can be overcome with a dual chamber valve.

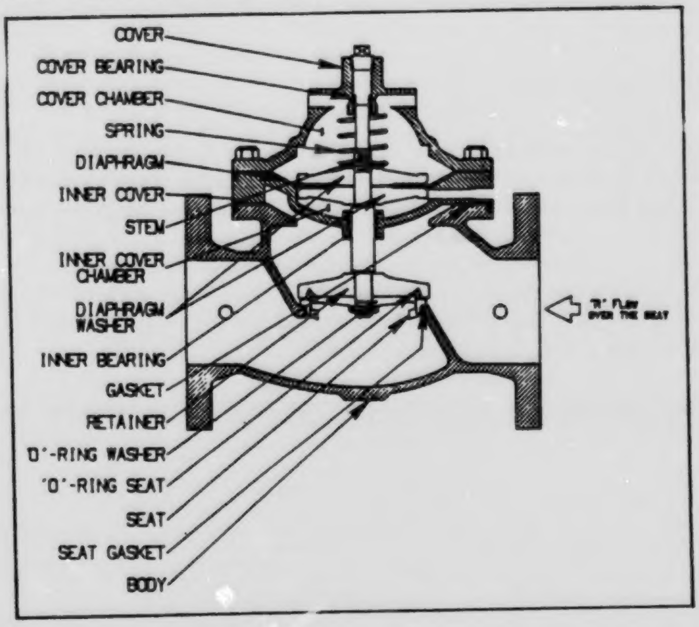

COVER
COVER BEARING
COVER CHAMBER
SPRING
DIAPHRAGM
INNER COVER
STEM
INNER COVER CHAMBER
DIAPHRAGM WASHER
INNER BEARING
GASKET
RETAINER
'O'-RING WASHER
'O'-RING SEAT
SEAT
SEAT GASKET
BODY
"R" FLOW OVER THE SEAT

FIGURE 500

A dual chamber valve adds a second isolated chamber under the diaphragm so that other than inlet or outlet pressure can be introduced. This 'third' pressure is usually either atmospheric (zero gauge) or pump inlet pressure. Since the lower chamber insert includes a bearing (with O-ring seal) the seat bearing is omitted. Now the valve can go fully open since we no longer need a pressure drop across it to ensure closing. We also find that even with a steep pump curve and sharp increase in inlet pressure on closing the closure rate will remain nearly constant. Dual chamber valves should be considered for any in-line pump application.

ANGLE PATTERN

Angle pattern valves are often viewed with a bit of mystique - probably because they look different from the conventional in-line or 'globe' pattern valve. In fact the angle pattern valve is identical to the globe pattern in terms of operation and uses the exact same components, except for the bottom inlet main body. Their main advantage is that with certain piping arrangements they fit in well. One should be aware that mounting any automatic valve with the stem horizontal will make service more difficult, particularly with large valves.

FIGURE 1100

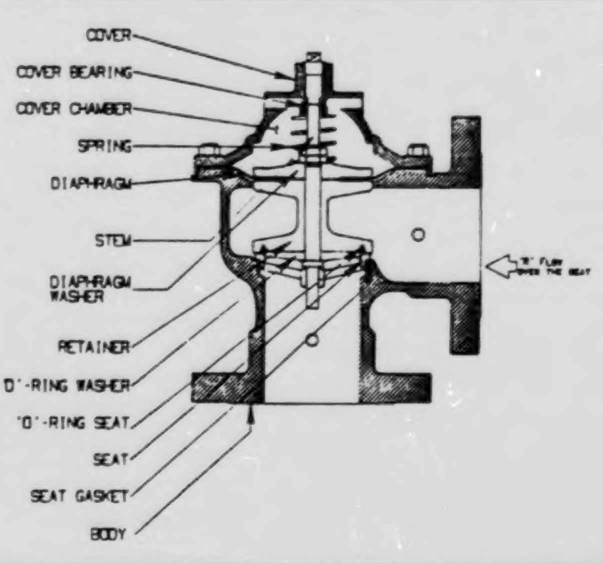
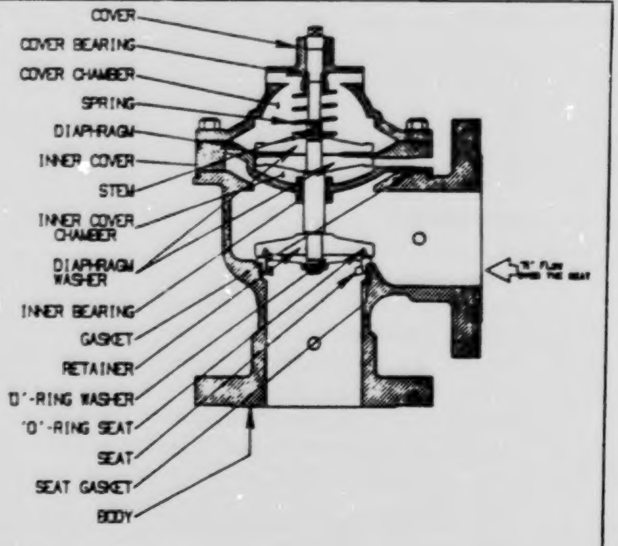

COVER
COVER BEARING
COVER CHAMBER
SPRING
DIAPHRAGM
STEM
DIAPHRAGM WASHER
RETAINER
'O'-RING WASHER
'O'-RING SEAT
SEAT
SEAT GASKET
BODY

FIGURE 1500


COVER
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COVER CHAMBER
SPRING
DIAPHRAGM
INNER COVER
STEM
INNER COVER CHAMBER
DIAPHRAGM WASHER
INNER BEARING
GASKET
RETAINER
'O'-RING WASHER
'O'-RING SEAT
SEAT
SEAT GASKET
BODY

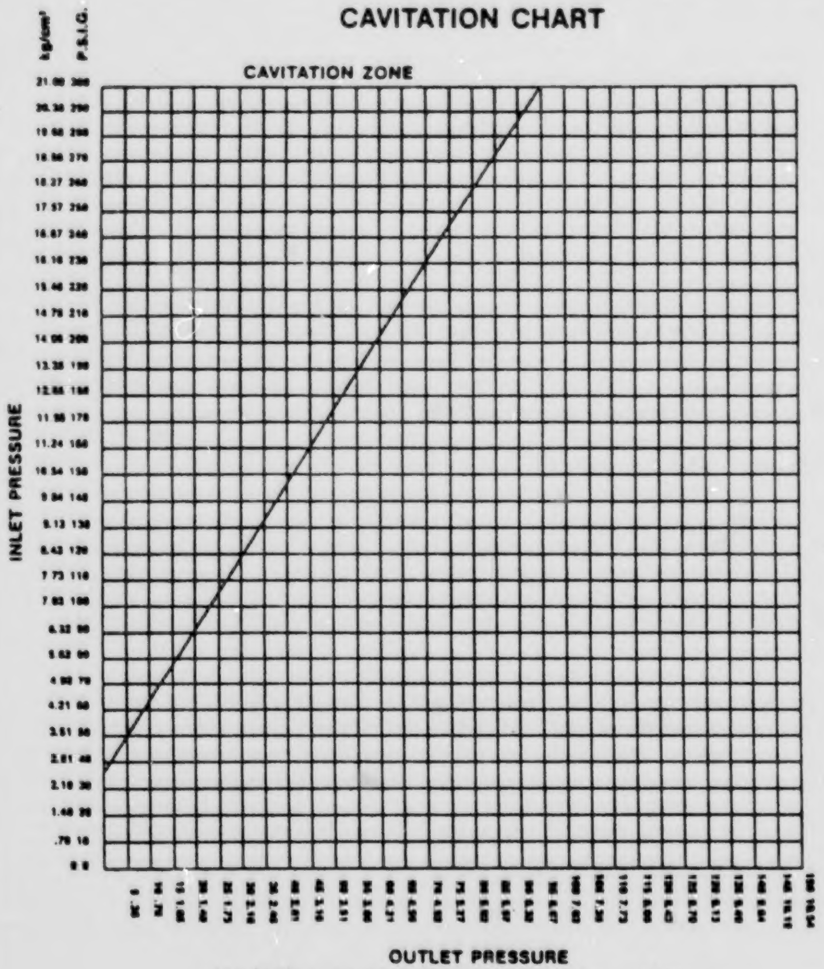
Angle valves do have somewhat less head loss when fully open since the bend to direct flow under the seat is eliminated. With our valves this difference is more pronounced than most, our angle valves having an average head loss of only 57% of globe valves due to the long bottom connection used to provide equal centre-to-end dimensions.

Appreciate that with modulating valves this lower full-open head loss is of no real advantage. While direct acting valves may appreciate an advantage in full-open pressure loss the primary consideration should be the piping arrangement involved.

There are four factors that may very significantly influence the performance and service life of a valve.

CAVITATION

We know that as the absolute pressure on water falls the temperature at which boiling occurs will also fall. At a low enough pressure even cold water will boil. Further we know that if the velocity of a fluid is increased by a nozzle its pressure will fall. Together these two phenomenon explain why cavitation (cold water boiling) can occur at a valve seat. The tendency for cavitation to occur is increased as the differential across the valve, hence seat velocity, increases. It is also more likely to occur with lower downstream pressures. Other than a tendency to 'choke' the valve the formation of water vapor is not a problem, but the sudden implosive collapse of vapor bubbles as the pressure rises back above the boiling level can, if near metal, duplicate an attack by a ballpeen hammer!



After selecting valve size, locate inlet and outlet pressures on cavitation chart. If point located falls in shaded area, cavitation can occur

VELOCITY

The faster a liquid flows through a valve the more turbulence will cause wear and noise. Guidelines have been established based on experience that, while certainly not absolutes, if respected will generally avoid unacceptable wear or noise. These are based on equivalent pipe size velocity.

Maximum continuous (velocity in feet per second) - 20 FPS
Intermittent (continuous service valve) - 25 FPS
Intermittent (standby service valve) - 45 FPS

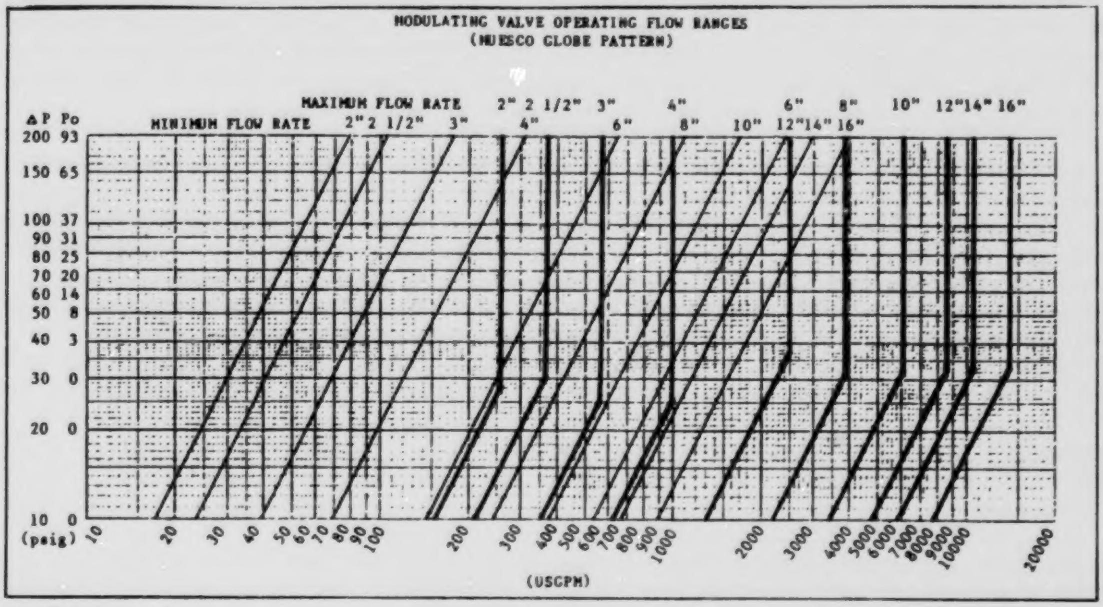
Valve Size	Flowrate (USGPM) for	20 FPS	25 FPS	45 FPS
1 1/4"		93	115	210
1 1/2"		125	160	280
2"		208	260	460
2 1/2"		300	370	650
3"		460	570	1000
4"		800	1000	1800
6"		1800	2500	4000
8"		3100	3900	7000
10"		4900	6000	11000
12"		7000	8600	16000
14"		8500	10500	19000
16"		11000	14000	25000

FLOW COEFFICIENT

When a modulating valve is operated very near its closed position localized seat velocity will be very high and wear in the form of the 'wire drawing' can be significant. Simply specifying a 'minimum flow rate' for each valve is just not good enough as the valve position needed to control at any flow, and thus wear, is greatly affected by the inlet-to-outlet pressure differential.

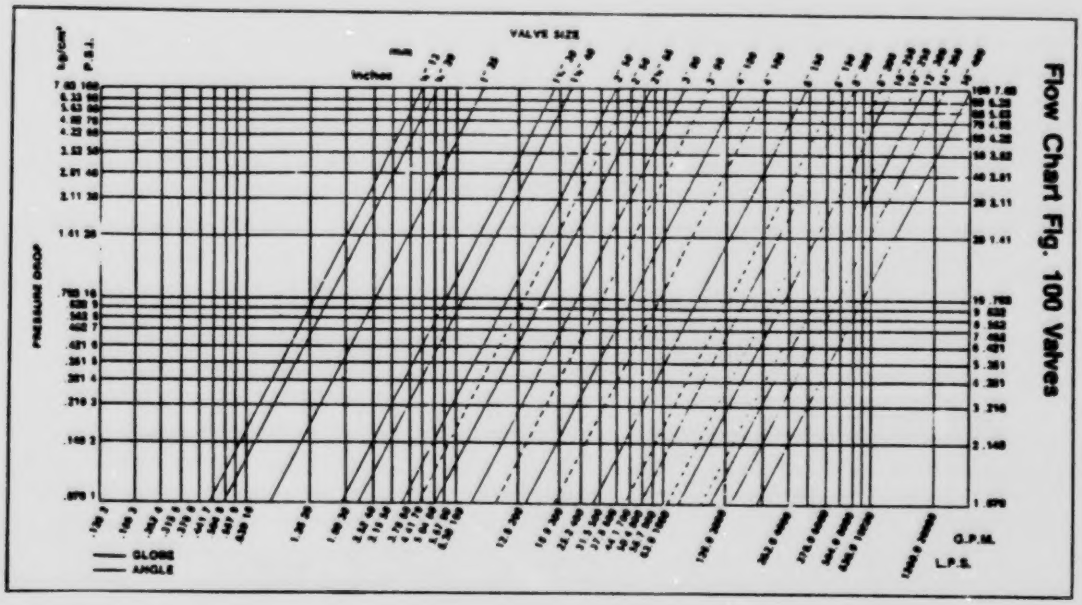
A much better approach is to limit minimum valve flow to 10% of the valve 'flow coefficient', or Cv, at given pressure differential, or Δ P. This lower limit can be calculated or the following chart can be used, which also gives maximum recommended flow, based on 25 FPS velocity or 90% of Cv (to ensure good response), as well as the minimum downstream pressure, Po, to prevent cavitation.

If operation at very low flows simply cannot be avoided the wear resistance of a valve seat can be significantly increased by the use of stainless steel seat material. Appreciate that the O-ring type seat seal design is inherently more resistant to low flow wear than a sharp edged seat design.



FRICTION

The friction loss (pressure drop) through a fully open valve is commonly provided, but it should be remembered that single chamber valves include a spring to ensure a minimum pressure differential is always established across the valve to provide a reasonable closing force. For this reason the minimum pressure drop is seldom much below 6-7 psig, particularly with smaller valves. In the case of dual chamber valves that can operate fully open, yet have positive closing, the friction loss varies only with flow and there is no minimum value to consider.



Flow Chart Fig. 100 Valves

By combinations of the various pilot devices available and by using valve stem actuated pilots and switches when needed it is possible to create literally hundreds of possible automatic valve variations. Discounting minor variations, impractical combinations and those of very limited application we are still left with perhaps a hundred or so useful variations.

The automatic valve is an extremely adaptable animal usually able to meet any functional requirement, or combination of functions. We consider ourselves to be in the business of selling valve functions rather than specific valves. Indeed the major concern is one of adequately and fully defining the problem so that the correct valve functions are provided. If conditions may occur that the valve has not been designed to control it is quite possible that physical damage may result to the system. Conversely if features are built into the valve that are really not required it may end up being overly complex and confusing to service.

CONTROL OF PRESSURE SURGE IN CENTRIFUGAL PUMP SYSTEMS

C.D. Smith
Professor of Civil Engineering
University of Saskatchewan

INTRODUCTION

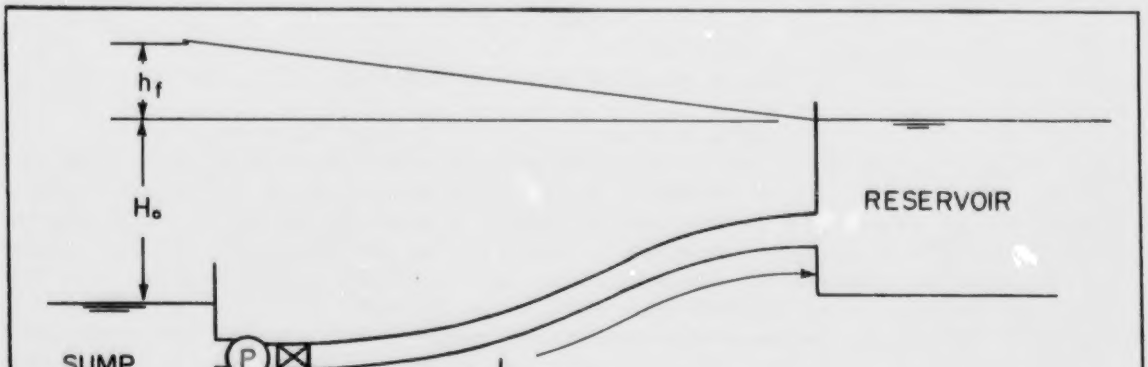
The continued development of water supplies for small communities and small irrigation projects requires the design and installation of appropriate pumping plants and distribution systems. Problems of surge and surge control may not receive the same attention as on larger projects, yet the problem is equally important if trouble free operation is to be obtained.

In this paper four particular methods of control of pressure surge arising from sudden power failure are discussed. The mechanics of the surge and its control are demonstrated using the graphical method of analysis.

Pressure surges result when the discharge at some point in a pipe flow system is suddenly changed. This can occur due to opening or closing a valve, or starting or stopping a pump. Surge due to these actions can be minimized by using positively controlled slow operating electrically driven valve and pump controls. In this way, the rate of change of discharge is regulated to a small value, and the corresponding pressure surge is likewise reduced. However, in the event of sudden loss of power, electrical devices become inoperative. This is a case which invariably occurs, often more than once a year, and it must be taken into consideration in the design. Sudden loss of power on a system operating at full capacity results in uncontrolled sudden stopping of a pump. Very often this is the worst case that must be considered in the design.

In this paper the case of a pump drawing water from a river, reservoir or well, and discharging into a water tower, canal or reservoir, is considered. The case is restricted to discharge into a single pipe of constant diameter. It is assumed that there is a check valve at the pump to prevent negative discharge and reverse rotation of the pump. This layout is shown schematically in Figure 1.

FIGURE 1
Definition Sketch



In Figure 1 the pump head H equals the static head H_0 plus the friction loss h_f in the discharge line. Usually H_0 is much larger than h_f, by design, but any combination of H_0 and h_f may be considered.

Upon sudden loss of power to the pump, a sub-normal pressure wave travels from the pump toward the discharge end of the pipe. The flow in the pipe will be stopped and reserved, and a positive pressure wave will travel down the pipeline toward the check valve at the pump. Alternating positive and negative surges will continue until damped out by friction. In extreme cases the negative pressure may drop to the vapour pressure of the water, producing parting of the water column, and the positive pressure may rise to several times the normal operating pressure. The possibility of such conditions arising must be checked and, where necessary, steps taken to reduce the magnitude of the surge. The methods considered in this paper are flexible pipe, flywheel effect, entrained air and air hoses. Other control devices that can be considered, but which are beyond the scope of this paper, include pressure relief valves, vacuum breakers, surge arrestors and compressed air surge tanks. Some of these devices may be used for special purposes or in combination with the methods discussed in this paper.

THEORY

The basic equation upon which all water hammer analysis is based (Pickford, 1969) are:

$$\Delta h = -\Delta V\ (a/g) \qquad (1)$$

$$a = \sqrt{K_w/\rho_w} \,/\, \sqrt{1 + K_w d/(Et)} \qquad (2)$$

and

$$t_c = 2L/a \qquad (3)$$

Equation 1 is Joukowsky's equation giving the water hammer head, or magnitude of the pressure surge, _ h, in terms of the magnitude of the velocity changes _ V, the pressure wave velocity a and acceleration due to gravity g. The negative sign is to account for the fact that the pressure increases when the velocity decreases, and vice versa. In the British system of units _ h is in ft, _ V and a are in ft/sec and $g = 32.2$ ft/sec^2.

In Equation 2, $\sqrt{K_w/\rho_w}$ represents the velocity of sound in water (4700 ft/sec), K_w is the bulk modulus of elasticity for water (300,000 psi), E is the modulus of elasticity for the wall material in the pipe, and d and t are the pipe inside diameter and wall thickness, respectively. Typical values of E for different pipe materials are given in Table 1.

TABLE 1

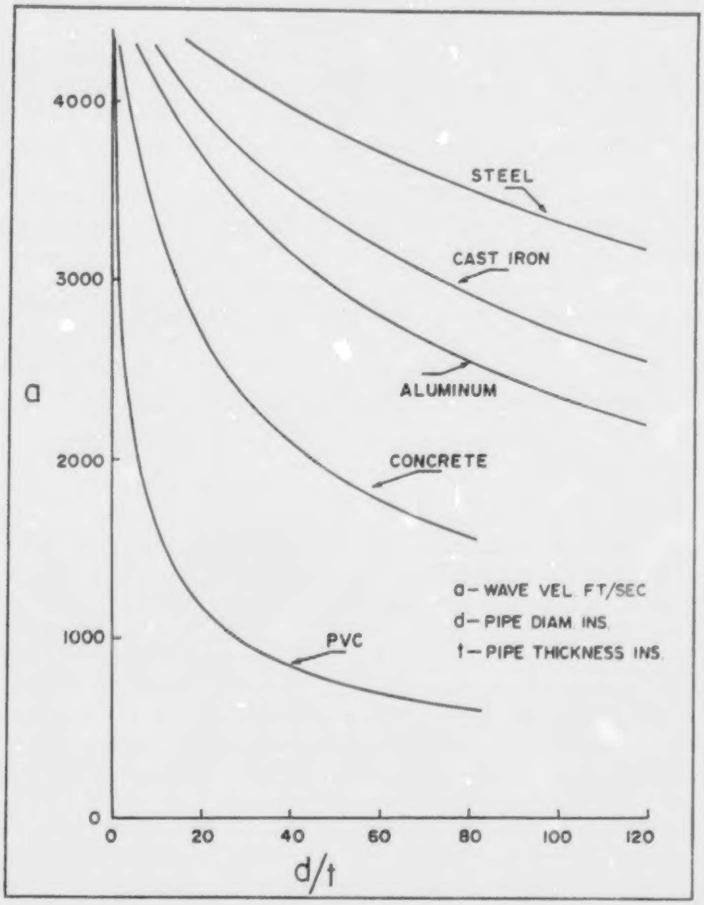
Typical Values of Modulus of Elasticity

Pipe Material	E in psi
Steel	30×10^6
Cast Iron	15×10^6
Aluminum	10×10^6
Concrete	3×10^6
PVC	0.4×10^6

A plot of wave velocity versus the d/t ratio is shown in Figure 2. All curves approach 4700 ft/sec as d/t → 0, corresponding to an inelastic pipe because of infinite wall thickness. Practical ranges of d/t are from 10 to 20 for PVC, 8 to 15 for concrete pipe, and 50 to 120 for metal pipes.

FIGURE 2

Effect of Pipe on Wave Velocity



Equation 3 gives the critical time t_c in terms of the pipe length L and wave velocity a. The critical time is the length of time taken for one round trip of the pressure wave. It is critical in the sense that the velocity change used in Equation 1 is limited to the change occurring during the critical time, since beyond this time the pressure is influenced by the arrival of reflected waves. It follows that larger pressure surges may occur for larger values of L because the increase in critical time may allow a greater velocity change before reflected waves modify the pressure wave pattern. Whether this occurs depends upon whether the pump stopping time is more or less than the critical time.

Calculation of pump stopping time is complex. The resisting torque which stops the pump after power is removed depends upon the type of pump, the pump speed (which is transient) and the pump head (which is also transient). The pump head also depends upon the proportion of the head that is static head and the effect of the induced pressure surges. Usually the pump stopping characteristics have not been determined in detail by the manufacturer and are not available from the supplier. Given the moment of inertia, speed, head and discharge, however, a stopping time can be calculated for any assumed resisting torque. For the simplest case, in which the resisting torque is assumed to be constant and equal to the steady state operating torque, the calculated stopping time would be

$$t_s = Wr^2 N^2 \eta/(183{,}400QH) \qquad (4)$$

in which t_s is the stopping time in seconds, W is the weight of all rotating parts in lbs, r is the radius of the gyration in ft, η is the pump efficiency, N is the pump speed in rev/min, Q is the pump discharge in ft^3/sec and H is the pump head in ft.

Actual stopping times are site specific and may be more or less than given by Equation 4. In the absence of any precise data on resisting torque or in situ testing of the pump, the equation may be used to estimate an approximate time.

Pump stopping times are surprisingly short. As an example (Anon. 1940), an 800 horsepower 1200 rpm pump at 20.9 ft^3/sec discharge and 284 ft head, with a Wr^2 of 1450 lb-ft^2, was observed to stop in 0.8 sec. Equation 4, using $n = 84\%$, gives 1.6 sec. If the numbers reported are correct, this is rather poor agreement. In the absence of more detail, no reason can be given for this difference.

GRAPHICAL ANALYSIS

The graphical method of solution for water hammer was the most practical and popular method from about 1930 to 1970 (Streeter and Wylie, 1967). Today computer methods are in common use, but the graphical method is still used. The advantages of the graphical method are that it is simple and easy to use, it does not require access to a computer or special software packages, and it is the only method that provides a visualization of the process as it occurs. This latter point is particularly useful for the novice. The writer believes everyone should clearly understand the graphical solutions before any attempt is made to write computer programs for water hammer problems. The graphical method may be used to illustrate how pressure surge is modified by changing pipe material, adding flywheel effect or using entrained air or air hoses.

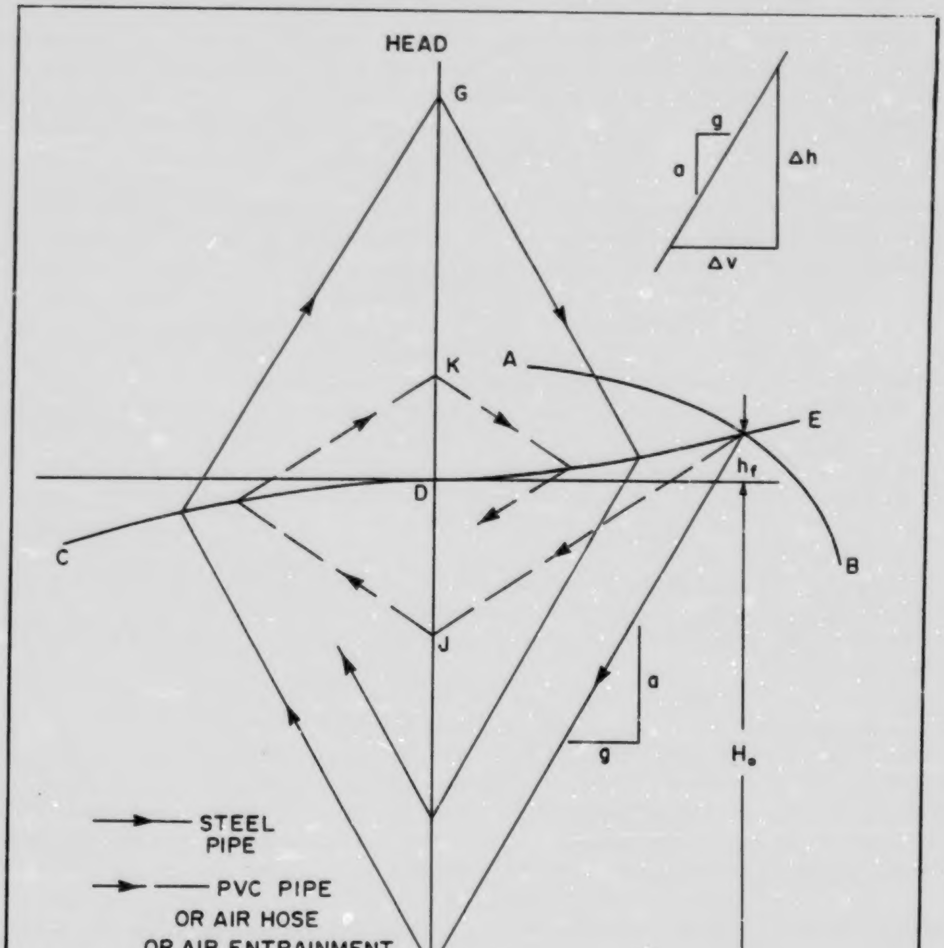
The essence of the graphical method is shown in Figure 3. It is basically a plot of head versus discharge, except the discharge is represented by the velocity in the discharge line. Line AB represents the head versus discharge characteristic curve at rated speed. Line CDE represents the head at the pump, consisting of the static head H_0 and the friction head due to flow in the pipeline, h_f. When the discharge is negative (flow to the left in Figure 1), the head at the pump would equal the reservoir head minus the friction loss, hence CD plots below the line. The intersection of AB and CDE locates the steady state operating point where the pump head equals the system head at the same discharge.

The triangle in the upper right of the diagram is a graphical representation of Equation 1, or

$$\Delta h \,/\, \Delta V = -\,g/a \qquad (5)$$

FIGURE 3

Graphical Solution for Pressure Surge


HEAD
G
g
a
Δh
Δv
K
A
E
D
h_f
C
B
J
a
g
H_o
STEEL PIPE
PVC PIPE OR AIR HOSE OR AIR ENTRAINMENT
F
O
−
VELOCITY
+

The sloping g/a line is used to determine the negative surge upon sudden power failure by extending it from the operating point to intersect the vertical axis at F. For purposes of this example, it is assumed that the pump has stopped in one t_c or less, the minimum pressure at F is still above the vapour pressure, and there is no reverse flow through the pump. When the reflected positive wave from the reservoir arrives at the check valve, there is a positive surge, given by the intersection at G. The surge diminishes with each pass of the wave until the final operating point is reached, at D. At D the discharge is zero and the head equals the static head.

FLEXIBLE PIPE

Assuming the solid line solution given in Figure 3 was for steel pipe with $a = 3600$ ft/sec, then if a thermoplastic such as PVC with $a = 1200$ ft/sec is used, then the g/a line would have a much flatter slope; in fact, one-third of the previous value. The solution for this case is shown by the dashed line in Figure 3. At once apparent is the reduction in negative and positive surge, to points J and K respectively. This illustrates how pressure surges can be reduced by using discharge pipe with a lower modulus of elasticity.

A word of caution is in order, however, because it does not automatically follow that surge will always be reduced by using more flexible pipe. If the pump stopping time is already longer than the critical time for steel pipe, then there may be little or no benefit in terms of surge reduction by going to a more flexible pipe. For example, if L is so small that $t_c = t_s/3$ for the steel pipe, then for the PVC pipe, t_c would equal t_s, and there would be a larger velocity change at the pump during the critical time interval. This would offset the effect of the flatter g/a line and minimize or eliminate the benefit. In other words, the ΔV in Equation 1 may be increased by as much as a is reduced. Each case must be judged on its own merit.

FLYWHEEL EFFECT

If the Wr^2 term in Equation 4 is increased by adding flywheel to the pump - motor axis, the pump stopping time can be increased. In effect, the unit will continue pumping, but at progressively reduced speeds, until the flywheel energy is exhausted. This will also reduce the magnitude of the surge.

The graphical solution including flywheel effect is shown in Figure 4. Several head versus discharge characteristics curves must be drawn for the pump - one for each critical time. Curve N_0 represents the head versus discharge when operating at rated speed N_0. Curve N_1 represents the head versus discharge when operating at speed N_1, where speed N_1 occurs at the end of one critical time interval. In the example shown, the pump is completely stopped in a time of $3t_c$, so curve N_3 is simply the vertical axis, for which $Q = 0$.

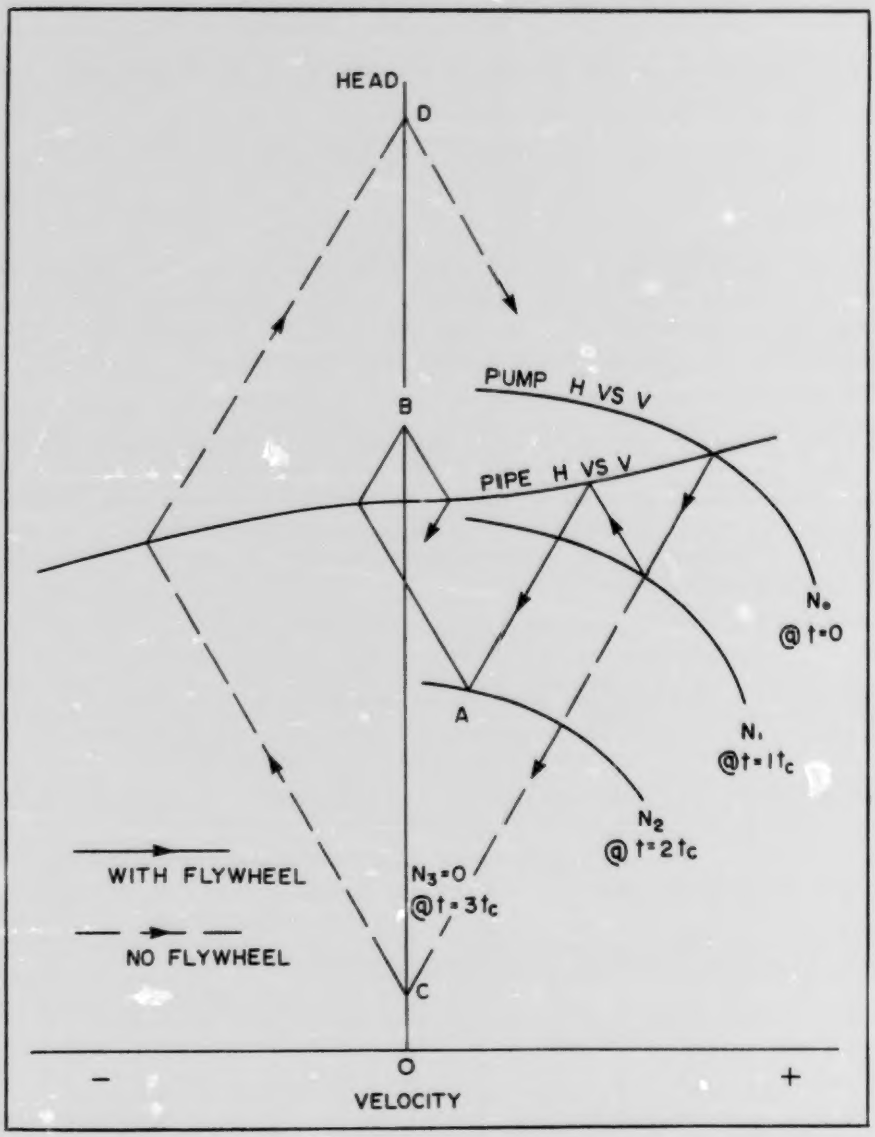
The negative surge at $t = 1t_c$ is found at the intersection of the g/a line and curve N_1. The construction proceeds giving the minimum negative surge at A and maximum positive at B. These compare to points C and D for the dashed line solution corresponding to no flywheel.

If the characteristic head versus discharge curve is known for speed N_0, the curves for any other speed can be easily calculated using the pump affinity laws. However, determination of the speeds N_1, N_2, etc. at each critical time interval is less simple. The speed-torque characteristics of the unpowered pump may not be known. In this case, it may be necessary to assume a linear variation in speed, starting at N_0 at time zero and proceeding to $N = 0$ at time t_s in Equation 4.

The advantage of the flywheel method is that it is foolproof. It does not depend on electrical, mechanical or hydraulic devices. On the other hand, the method is expensive, particularly for long pipelines. In order to make $t_s \geq 3t_c$ for a long pipeline, it may require a massive increase in Wr^2 and the cost could be prohibitive.

FIGURE 4

Graphical Analysis for Flywheel Effect


HEAD
D
B
PUMP H VS V
PIPE H VS V
N_0 @ t=0
N_1 @ t = 1 t_c
A
N_2 @ t = 2 t_c
N_3 = 0 @ t = 3 t_c
WITH FLYWHEEL
NO FLYWHEEL
C
O
−
+
VELOCITY

AIR ENTRAINMENT

If air bubbles are dispersed through the water, the pressure wave velocity for the air water mixture will be modified. It can be shown that for a gas-liquid mixture

$$a = (K_w/[1 + \beta(K_w - K_a)/K_a]/[\rho_w (1 - \beta) + \rho_a \beta]\ [1 + K_w d/Et/(1 - \beta(K_w - K_a)/K_a)])^{1/2} \qquad (6)$$

in which K_w and K_a are the values of modulus of elasticity for the liquid and gas, p_w and p_a are the densities of the liquid and gas, and B is the gas content ratio, by volume, as in

$$\beta = V_A/(V_A + V_w) \qquad (7)$$

If $\beta = 0$ (no gas), equation 6 reduces exactly to equation 2. If B = 1 (all gas), equation 6 becomes

$$a = \sqrt{K_a/Pa} \,/\, \sqrt{1 + K_a\, d/ET} \qquad (8)$$

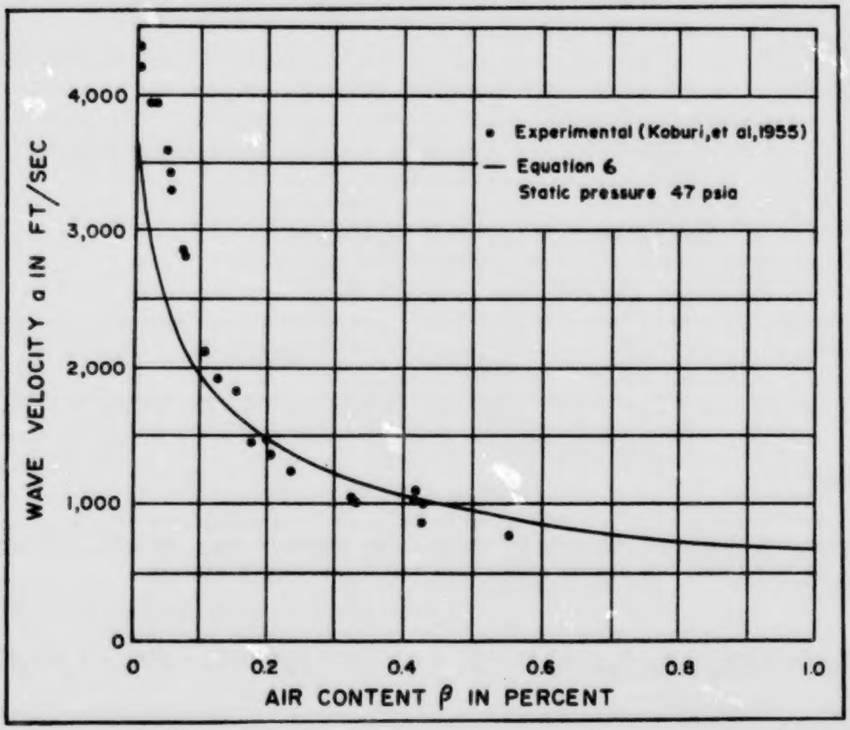
which must be the case.

The bulk modulus of elasticity for air is kp, where k is equal to 1.4 (the ratio of specific heats) and p is the absolute pressure. At atmospheric pressure, for example, $K_a = 1.4 \times 14.7 = 20.6$ psi. Since $K_w = 300{,}000$ psi, it is seen that the air is almost 15000 times more compressible than water. As a consequence, a surprisingly small amount of air will give a large reduction in wave velocity.

Equation 6 is shown plotted on Figure 5, along with some experimental data by Korori et al (1955). It is seen that an air content of only 0.2% by volume will reduce the wave speed to about one-third of the velocity of sound in water. The effect on pressure surge is comparable to using a more flexible pipe. In fact, the process is similar except that part of the volume change in the liquid is accommodated by contraction and expansion of the air bubbles instead of contraction and expansion of the pipe.

FIGURE 5

Effect of Entrained Air on Wave Velocity



The air entrainment system for reducing pressure surge requires that air be pumped continuously into the water just downstream from the pump in order to maintain the required value for B throughout the length of the pipeline. Only a small compressor is needed. Energy costs for operation are insignificant compared to the operation costs for the pump. However, some risk is involved, as with any mechanical device, in that the compressor may fail at an inopportune time. A second compressor on standby is needed to minimize this risk. A further limitation of the method is that in a long pipeline the diffused air may coalesce into an air pocket which may form at a high point in the pipeline.

AIR HOSES

The benefit of the air entrainment method without the limitations may be achieved by using sealed air filled flexible plastic or rubber hoses attached to the inside of the pipe along the crown. The hoses must not be reinforced with nylon or fabric because they must be able to readily expand and contract as the oscillating pressure surges travel up and down the pipeline. A relatively thin walled soft rubber or tygon tubing is satisfactory. Hoses should be about 10 or 15 ft in length. They should be sealed at an internal air pressure equal to about two-thirds of the normal pipeline operating pressure. It should be noted that a single hose for the whole length of the pipeline cannot be used because all the air would migrate toward the point of lowest pressure in the line, such as at a high point or at the downstream end of the pipeline.

During operation, the air pressure in each hose is essentially equal to the water pressure in the pipeline at the same position. At normal pressure, the hoses will be oval shaped to account for the fact that the air volume will be less than the volume corresponding to the initial sealing pressure. During a pressure drop, the air will expand and the hoses will become round and stretch, and during a pressure rise, the hoses will flatten to whatever extent is necessary to accommodate the reduced air volume. In essence, the performance is similar to the entrained air method, and in fact would be identical if it were not for the stress in the wall of the hoses.

Figure 5 may be used to estimate the size of air hose needed to achieve a particular wave velocity. If 0.2% air volume in the cross section is desired, an air hose with an inside diameter of 4.5% of the pipe diameter would be required. This would require a 1.6 inch hose in a 36 inch pipe, for example.

The air hose method eliminates the need for an air compressor and avoids any problems of air accumulation in the pipeline. The hoses would have to be inspected periodically so that any defective lengths can be replaced.

CLOSURE

This paper has drawn attention to four methods of reducing pressure surges in a pipeline following sudden power failure. The mechanism by which the surge reduction occurs is explained, and the method of analysis is illustrated with particular reference to the graphical method.

In the examples shown, it was assumed that the pump operating pressure minus vapour pressure was greater than the pressure drop due to negative surge, thus avoiding parting of the water column. However, parting of the water column can also be analyzed using the graphical method (Stepanoff, 1957). Similarly, reverse flow with or without reverse pump rotation can be analyzed for the case of no check valve. This requires knowledge of the reverse flow and turbine characteristics of the pump. The methods described here still reduce the magnitude of the surge, but the final operating point would be between C and D on the CDE curve on Figure 3.

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ULTRAVIOLET SPECTROPHOTOMETER AND GAS CHROMATOGRAPH AS TOOLS FOR MONITORING ORGANICS AND CONTROLLING TASTE AND ODOUR

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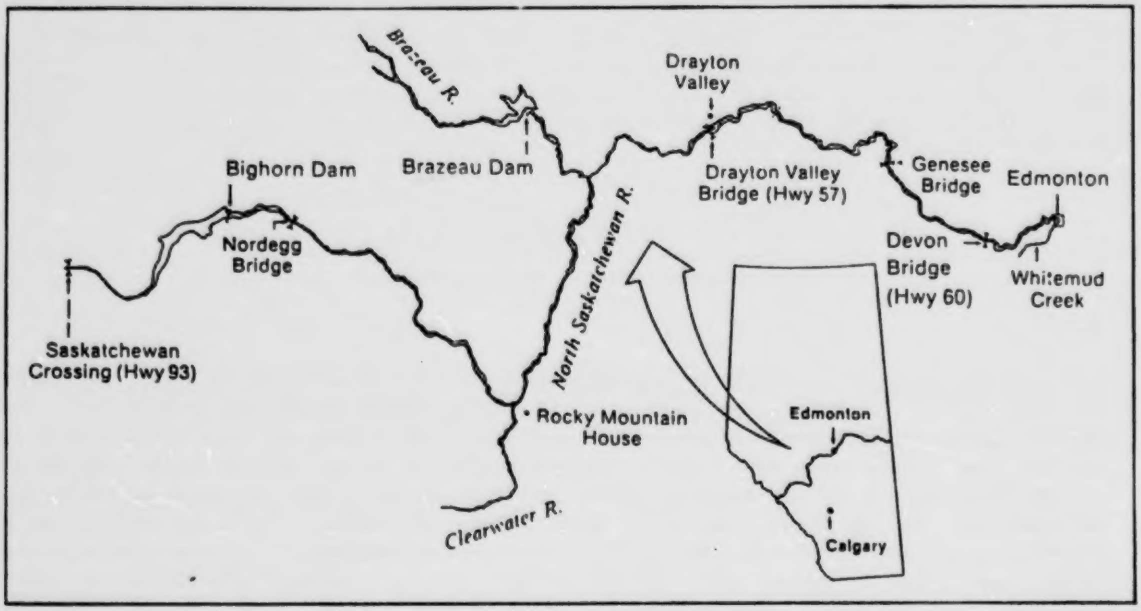
The North Saskatchewan River is the water supply for the City of Edmonton (see Figure 1). It originates in the Saskatchewan Glacier in the Columbia Icefields approximately 500 km west of Edmonton. Major tributaries include the Clearwater, Brazeau and Nordegg Rivers.

FIGURE 1

The North Saskatchewan River Basin Upstream of Edmonton

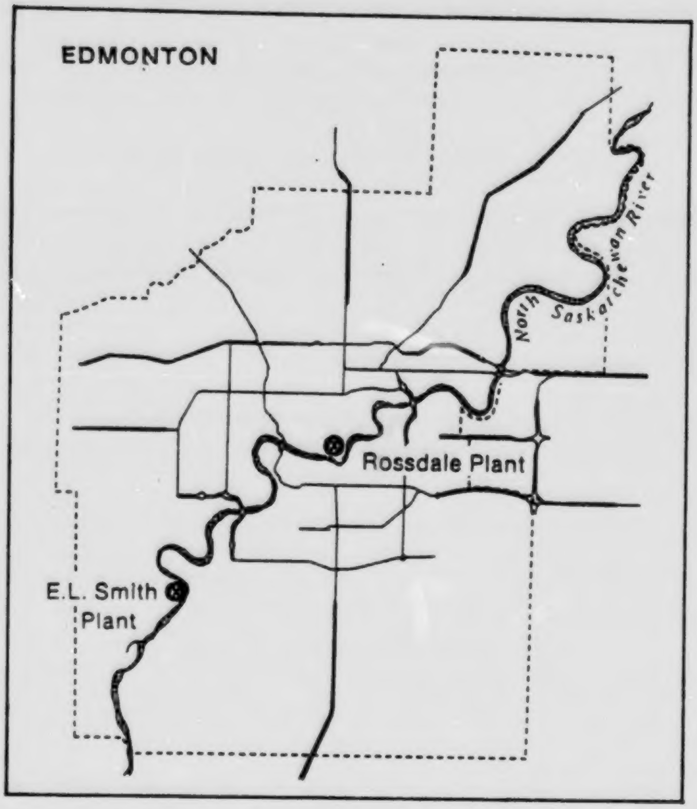

Brazeau R.
Drayton Valley
Bighorn Dam
Brazeau Dam
Drayton Valley Bridge (Hwy 57)
Genesee Bridge
Edmonton
Nordegg Bridge
North Saskatchewan R.
Devon Bridge (Hwy 60)
Whitemud Creek
Saskatchewan Crossing (Hwy 93)
Rocky Mountain House
Edmonton
Clearwater R.
Calgary

The Edmonton Department of Water and Sanitation has four water treatment plants which serve a population of 700,000. As shown in Figure 2, the department operates water treatment plants at two sites on the river, the Rossdale Plant in the centre of the City (comprised of three separate plants namely Plant No. 1, 2 and 3) and the E.L. Smith Plant in the Southwest quadrant of the City (Plant No. 4).

At its origin the glacial melt water is of good quality, but the water from adjoining tributaries carry substantial organic loading. Several smaller streams, often with even higher organic content, also join the North Saskatchewan River (NSR) just upstream of Edmonton.

FIGURE 2

Location of Edmonton Water Treatment Plants



Runoff

There are two major annual runoff periods experienced in the NSR: early spring runoff and summer mountain runoff. The spring runoff can be divided into upstream runoff and urban runoff. The spring runoff results from local melting of large quantities of snow and ice (including that in the snow dumps). The Urban runoff within the city limits has a significantly deleterious impact on raw water quality of the Rossdale plant, particularly with regard to micro-organisms and hydrocarbons. In the spring, the urban runoff affects the Rossdale Plant exclusively, while upstream runoff affects both E.L. Smith and the Rossdale plants equally. Runoff from upstream rural areas poses a concern with natural organic matter, commonly referred to as humic substances, arising chiefly from decaying vegetation in muskeg areas.

Summer mountain runoff conditions are experienced between May and June. The degree of severity depends on the amount of rain that accompanies the mountain snowfield melt.

Taste and Odour

The taste and odour problems, in Edmonton's drinking water during each of these runoff periods are distinct. From combined ultraviolet (UV), gas chromatograph (GC) and odour-monitoring data the following groupings of taste and odour complaints have been established.[1]

1. Hydrocarbons - diesel, ethylbenzene, napthalenes, and so forth can cause taste and odour at extremely low levels. Generally a result of urban runoff.

2. Chloramines - used at higher levels can produce a strong chlorinous or bleachlike taste.

3. Musty or earthy odours - during runoff events, the river carries drainage from decomposing organic matter which may have collected on the land. These materials can cause musty or earthy odours in the water. Also, ice break-up can release actinomycete metabolites [geosmin] into the water from river—bank soil.

4. Metabolites of Algae - algae growth can give rise to a variety of taste and odour problems (including "fishy" odours).

A study of the possible causes of tastes and odours associated with spring runoff in Edmonton drinking water was undertaken by the Pollution Control Division of the Alberta Department of the Alberta Environment in 1982. In its report,[2] the Division concluded that storm sewers are the major source of poor quality raw water at Rossdale during spring runoff. Major synthetic organic components identified in one of the main storm sewer effluent streams discharge upstream of the Rossdale Plant intake were methyl, ethyl and dimethylethyl benzenes; and methyl and dimethyl naphthalenes, naphthalene, and straight and branched alkanes and alkenes.

The City and Province jointly commissioned Professor Steve Hrudey from the University of Alberta to assess the quality of Edmonton's drinking water with specific emphasis on taste and odour[3]. The Hrudey study carried out an extensive study of the North Saskatchewan River and concluded that during urban runoff substantial quantities of hydrocarbons were discharged from the storm sewers and these similar types of hydrocarbons were present in the raw water at the Rossdale intake. The Hrudey[3] study outlined various sources and discharge types and these are summerized under Table I.

TABLE I

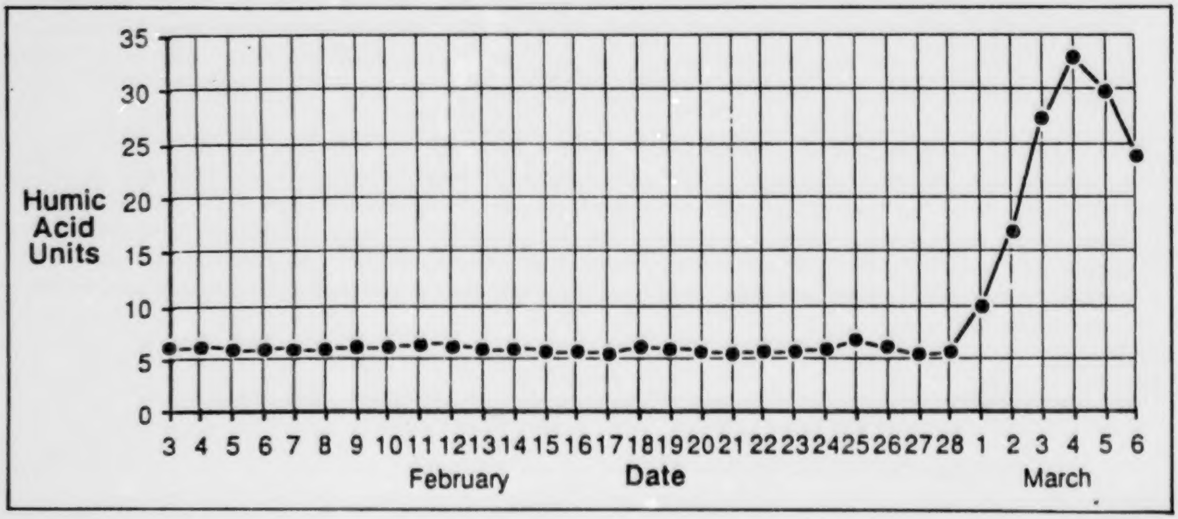
SOURCE:	TYPE OF DISCHARGE:
Direct Point Source Discharge	
Rocky Mountain House	Sewage
Drayton Valley	Treated Municipal and Industrial
TransAlta Utilities - power plant.	Waste dissolved solid.
Devon	Chronic hydrocarbon contamination
Indirect Point Source Discharges	
Several small municipal sources	Municipal and industrial waste
Wabamun Power Plant	Wastewater.
Runoff Discharge - Edmonton	
7 Storm sewer contribution	Surface runoff and raw sewage.
22 Storm sewer outfalls	Some quantities of sanitary sewage
Unintentional cross connections	Sanitary Sewage
Storm runoff (Reynolds and Livingston Survey)[2]	
Periodic Spills	Oil, hydrocarbons
Snow dump	Hydrocarbons
Natural Organic Carbon	
Forest fallen leaves, decaying Vegetation.	Soil humas.

Monitoring of Soluble Organic Carbon of Molecular Weight Greater than 500 daltons

Water from the North Saskatchewan River contains measurable quantities of organic carbon from the sources indicated in Table I. When concentrations of humic substances become high enough, the water will display visible colour. Thus water from a swamp or from a muskeg region displays a yellowish brown colour which is caused by humic like materials. The soluble organic carbon can be measured by (i) dissolved organic carbon (DOC), (ii) colour, which is a measure of the visible light absorbed by the water containing humics and (iii) UV light absorbance, which appears to provide a better range of sensitivity. In the springtime, the soluble organic carbon levels in NSR Edmonton are higher than at other times of the year. (See Figure 3).

FIGURE 3

Relative Concentration of Humic Acids (UV) in Raw Water at the Rossdale W.T.P. (Feb 3 to March 6)



UV as a Tool for Monitoring Organics

In 1985 a severe spring runoff season impacted all plants for a few weeks. In March of that year the water treatment plant laboratory began to examine the UV absorbance analysis as a tool for monitoring humic like substances.

The UV method is very simple. It involves the filtering of distilled water as blank, treated water and raw water through a 0.45 micrometer cellulose acetate membrane filter and the measurement of the absorbance at 200 nm using a 4 cm quartz cuvette. The absorbance is converted into humic acid units (HU) from an absorbance vs concentration curve, using a commercially available humic acid as standard reagent. To prevent breakage in the handling of the quartz cuvettes, a flow through cuvette has been adapted to the spectrophotometer as shown in Figure 4.

A typical scan of the raw and treated water from the E.L. Smith Plant before and after the runoff is shown in Figure 5.

FIGURE 4
Adaption of a Flow Through Cuvette in the Spectrophotometer

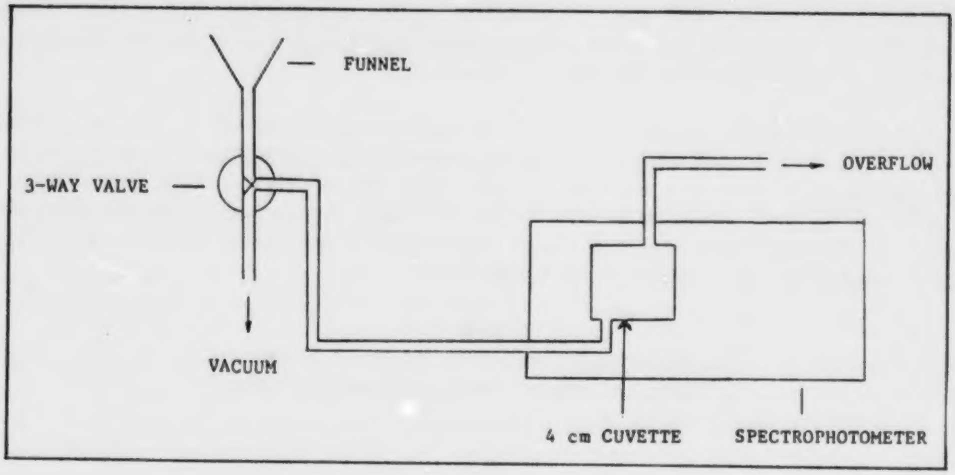
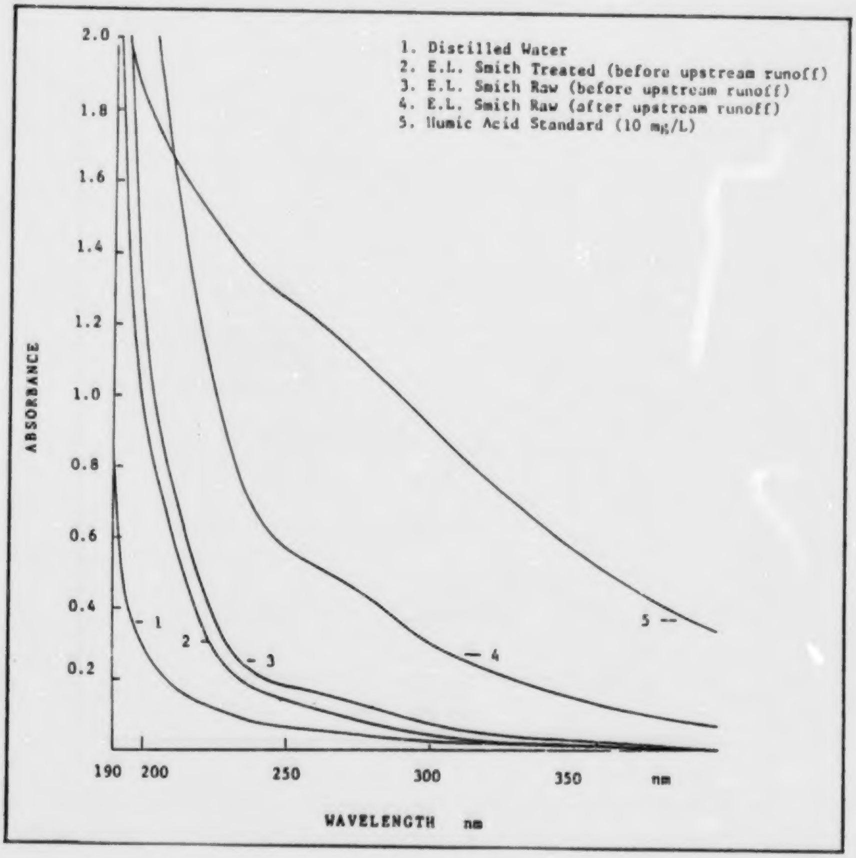

FUNNEL
OVERFLOW
3-WAY VALVE
VACUUM
4 cm CUVETTE
SPECTROPHOTOMETER

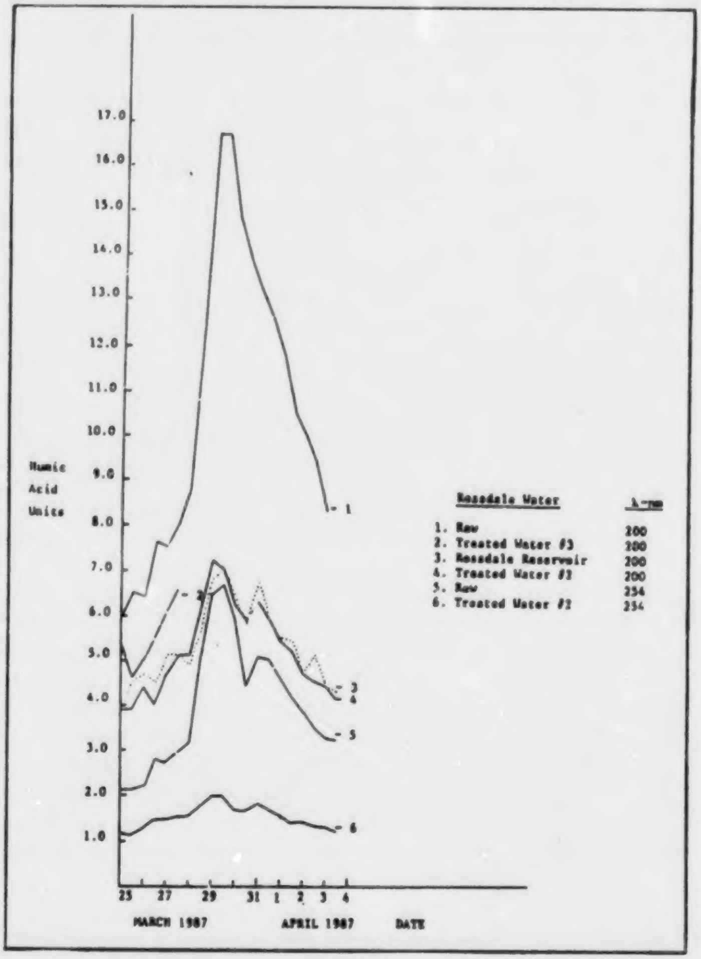
FIGURE 5
Ultraviolet Absorption Spectra of
Water Samples and Humic Acid Standard



Humic acid is certainly not an ordinary natural product. It is defined as the fraction of soil organic matter which is soluble in dilute base, but insoluble in mineral acid and alcohol. Attempts to characterize humic acid by sophisticated spectrophotometric methods, such as infrared and ultraviolet have been unsuccessful. The ultraviolet and visible spectra are monotonic with no observable bands and indicate only high absorption in the region below 400 nm. The observation can be seen from the scan in Figure 5.

The City of Edmonton opted to conduct the UV analysis using 200 nm and a 4 cm cell, because the difference between "poor raw water" and "good treated water" absorbance is observed to be the greatest at 200 nm. Some work was carried out using 254 nm with a 10 cm path length, but as shown in Figure 6, both curves have more or less the same shape except that the absorbance peak is greater at 200 nm. Measuring the absorbance at 200 nm has proven to be very practical, as additional sensitivity is obtained at this wavelength.

FIGURE 6
Relative Concentration of Humic Acids
in Raw and Treated Water at Rosedale Plant



Many water treatment plants in Germany and the Netherlands use UV for process control; however their analysis are based on a UV absorbance at 254 nm[7,9]. For specific waters it is possible to establish a good relationship between DOC and UV absorbance and because UV can be measured continuously and on-line; its process control application is obvious.

UV Application at Plant Scale

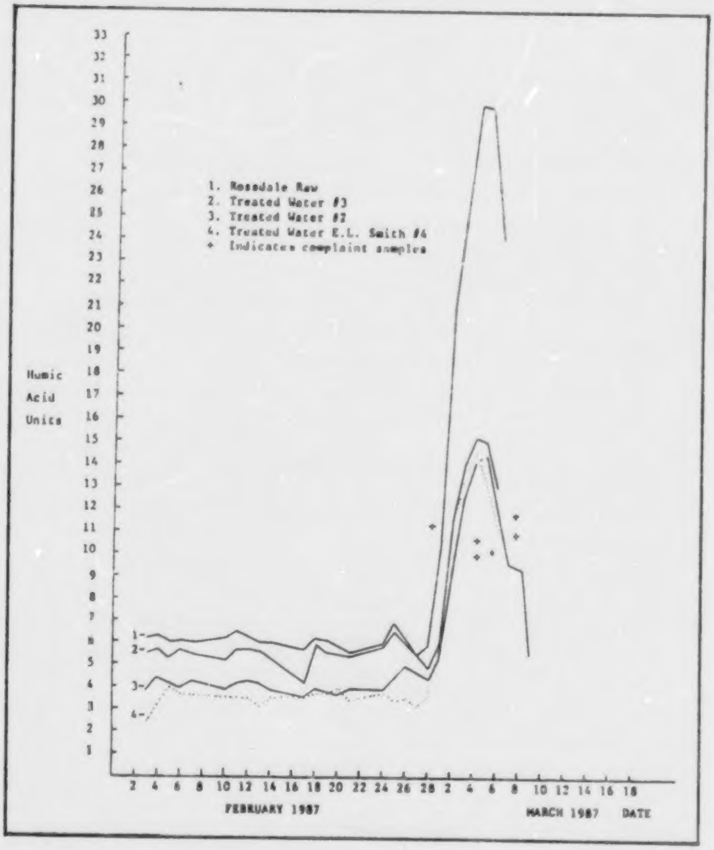
With the success of the UV absorbance test for monitoring water quality conditions in Edmonton's water, the water treatment plant laboratory set out an in-house study. Following were the objectives and their respective outcome.

Objective 1: To observe the organoleptic quality of treated water through consumer complaints and to determine quantitatively objectionable tastes and odour using humic acid as a standard.

Conclusion: UV absorbance was measured on tap water samples obtained from sources of consumer complaints. By superimposing complaint data on the treated water data, Figure 7, it was established that when the treated water readings were above 10 HU (Humic Acid Units), consumer complaints occur.

FIGURE 7

Relative Concentration of Humic Acids in Raw Water and Treated Water at Rosedale and E.L. Smith Plants



Objective 2: To monitor the plants (Rossdale Plants #1, 2 and 3 and E.L. Smith) and determine their respective efficiency in removing humic substances.

Conclusion: From Figure 7, it can be clearly seen that the E.L. Smith Plant demonstrates the best removal efficiency while Rossdale Plant #3 is the least efficient in removing humic substances. The E.L. Smith plant is relatively new (1977) and its process involves a two stage alum clarification lime softening treatment with a total treatment time of 7 hours. Plant #3 is a small packaged plant ; its process can only carry out a one stage combined alum clarification and lime pH correction process, with a retention time of under three hours.

Objective 3: To develop an operating criteria for the treatment of taste and odour using powdered activated carbon (PAC) at various UV absorbance (Humic acid unit) concentrations scenarios.

Conclusion: A table of summary of operations response to taste and odour treatment at the E.L. Smith Plant is shown under Table 2. The criteria was developed after a two year experience with treating water with potential taste and odour causing characteristics with simultaneous monitoring of UV absorbance.

TABLE 2 - E.L. Smith Water Treatment Plant - Taste and Odour Control Procedures

DATE: 23/Jun/87

DESCRIPTION OF CONDITION	PROCESS STATUS					MONITORING RESPONSE				CHEMICAL FEED RESPONSE		OTHER RESPONSE
	Unit	Raw	Eff1 C1/C2	C3 Eff	Potable	** GC	UV/Color	Odour	Jar Tests	Carbon	ClO2	
1 PRECAUTIONARY MEASURE Feb. 15 - Oct. 15	UV Color Turb VO	<10 <15 <50 <5	\/ /\ \/ /\	\/ /\ \/ /\	\/ /\ \/ /\	1 Raw/day 1 Trt/day	2 Raw/Sh 2 Trt/Sh	6/Shift	As Required	2 ppm	0.10 - 0.19 and Total ox<0.4	See conditions below for specific response.
2 TURBIDITY (NTU)	Turb Turb Turb	>50 >50 >50	- - -	- >15 -	- - >1	1 Raw/day 1 Trt/day	3 Raw/Sh 3 Trt/Sh	6/Shift	At least 2 per shift or when Turbidity Change >10	5 ppm	0.10 - 0.19 and Total ox<0.4	Step-up clarifier monitoring Plant to Waste until Recovery* Plant to waste*
3 UV ABSORPTION (Humic Acid Units)	UV UV UV UV	>15 >15 >15 >15	<5 >5 >10 >10	<5 >5 >10 >10	<5 >5 <10 >10	Potable > 8 2 Raw/day 4 Trt/day	3 Raw/Sh 2 Int/Sh 3 Trt/Sh	12/Shift	At least 3 per shift with UV as criteria for Jar Test.	5 ppm 10-15 ppm 15-20 ppm 25 ppm	0.10 - 0.19 and Total ox<0.5	Monitor closely Step-up monitoring Step-up monitoring Plant to waste* cut pumpage
4 COLOUR (TCU)	Color Color Color Color	>20 >20 >20 >20	<2 >2 >4 >5	<2 >2 >4 >5	<2 >2 <5 >5	Potable > 5 2 Raw/day 4 Trt/day	3 Raw/Sh 2 Int/Sh 3 Trt/Sh	12/Shift	At least 3 per shift with color as criteria	5 ppm 10-15 ppm 15-20 ppm 25 ppm	0.10 - 0.19 and Total ox<0.5	Monitor closely Step-up monitoring Step-up monitoring Plant to waste* cut pumpage
5 GC (VO) (Volatile organics - ppb)	VO	>2 - >20 -	- - - -	- - - -	- 3 - 5 - >10	Potable > 8 4 Raw/day 4 Trt/day	2 Raw/Sh 2 Int/Sh 2 Trt/Sh	12/Shift	As Required	10 ppm 15 ppm 15-20 ppm 25 ppm	0.10 - 0.19 and Total ox<0.5	Step-up monitoring Step-up monitoring Step-up monitoring Plant to waste* cut pumpage
6 GROSS POLLUTION REPORTED (Chemical or Oil Spill)	N/A					4 Raw/day 4 Trt/day	6 Raw/Sh 2 Trt/Sh	12/Shift	As Required	15 ppm	0.10 - 0.19 and Total ox<0.5	Step-up Raw Water monitoring Once spill is detected, follow above conditions to respond..
7 VISUAL OBSERVATION OF OIL IN PLANTS	N/A					4 Trt/day	8 Trt/Sh	12/Shift	As Required	>15 ppm	0.10 - 0.19 and Total ox<0.5	Step-up Treated Water monitoring: When condition identified follow above criteria to respond.
8 INCREASED CHLORINE DIOXIDE DEMAND Not Meeting Residual Due to Total Oxidants	N/A					2 Raw/day 4 Trt/day	3 Raw/Sh 3 Trt/Sh	12/Shift	As Required		Total Oxidant <0.5	Increase monitoring of all parameters. Ask Lab to increase frequency of yield determinations.
9 ODOUR	Odour is detected in treated water					Call Lab to do immediate GC.	6 Raw/Sh 6 Trt/Sh	12/Shift	As Required	25 ppm	0.10 - 0.19 and Total ox<0.5	If field and site reservoirs in full condition, consider wasting.*

Objective 4: To provide a sensitive early warning system to the two plant sites to indicate the sudden increase of humic substances in the river water approaching the City.

Conclusion: A spectrophotometer was installed upstream at Devon 18 kilometers from Edmonton and the results were made available to both plant sites on a regular basis. The trend of observance is as shown in Table 3. This monitoring provided the E.L. Smith Plant about 3.5 hours of early warning and Rossdale plant 7.0 hours early warning, on measurements done on the water at Devon. In both plants the warning was adequate to initiate process changes in response to upstream conditions. Plans are now underway for early next year to install an online UV monitor at Devon, with signals reaching both plants on a continuous basis.

TABLE 3

PLANT	DATE	TIME	HU UNITS	(24 Hours Ago)
Devon	April 1	Noon	21.3	(8.1)
E.L. Smith	April 2	7:00 am	16.6	(7.9)
Rossdale	April 2	9:50 am	17.6	(9.9)

Objective 5: To monitor the distribution system reservoirs in order to determine any fluctuations in concentration of humic substances.

Conclusion: From the UV data on various field reservoir samples, it was observed that treated water in the reservoirs has a higher level of humic like substances late in summer and early autumn. This effort of monitoring will, in the future, enable the City to prevent the build up of these substances in the reservoirs and may also provide a basis for determining reservoir cleaning schedules which are currently carried out every five years irrespective.

Objective 6: To monitor humic acid removal efficiency through the treatment processes train to observe the most effective treatment process/chemicals.

Conclusion: The UV method was used to monitor the water treatment process at various stages. The results so far have been inconclusive although indications are that it could be utilized to optimize the treatment process for the removal of humic substances, if good data is collected over several seasons and severe spring conditions.

Objective 7: To investigate and introduce additional measuring parameters to deal with taste and odour control in Edmonton's drinking water.

Conclusion: The achievement of the above objectives clearly demonstrated the usefulness of UV and it was incorporated as one of the routine monitoring parameters in 1986.

Future Plan:

Plans are now underway to use UV to monitor pilot-plant performance for removal of non purgeable total organic carbon (NPTOC) and THM precursors.

The other parameter that was also investigated was gas chromatographic analysis and this is described below.

GC as a Tool for Monitoring Volatile Organics

In 1977, the water treatment laboratory purchased a gas chromatograph (GC) with a purge and trap option and flame ionization detector (FID). The successful application of GC monitoring resulted in the purchase of GC with purge and trap for plant operators in 1986.

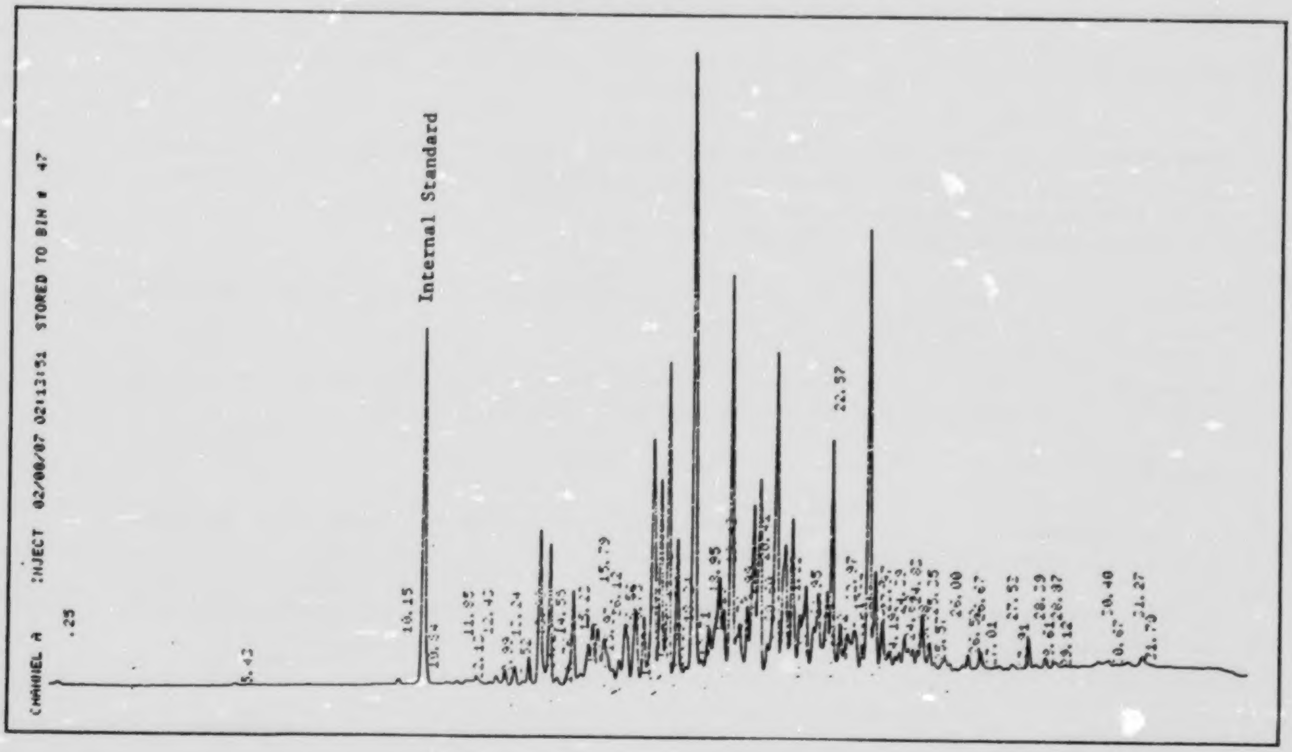
These GC's proved to be of immense value and were employed for the following applications:

1) RUNOFF

They were employed to monitor raw and treated water for trace organic volatiles round the clock. As described earlier, during urban runoff caused by local rain, raw water at the Rossdale Plant has the presence of hydrocarbons. The plant has developed an action plan which calls for the progressive addition of PAC in response to increasing levels of volatile organics. Every effort is made to see that the level of volatile organics leaving the plant does not exceed 10 ppb. An early warning system, recently installed on the GC, automatically indicates when raw or treated water exceeds 8 ppb.

Through the observation of organoleptic quality of treated water and through complaints received as an indicator, it has been determined that when treated water volatile organics exceed 10 ppb, consumers will complain of taste and odour problems. In the extreme, the plan can lead to a plant shutdown in response to very bad water conditions. Figure 8 indicates a chromatogram for a raw water sample taken from the Rossdale Water Treatment Plant at the beginning of urban runoff.

FIGURE 8
Gas Chromatogram of Rossdale Water Sample



2) MAINTENANCE OF PLANTS

After maintenance of a plant's units and components is carried out it is a routine practice to check for the presence of volatile organics. On many occasions oil and grease presence can trigger the GC alarm. The treated water is not put through the distribution system until the level of total volatile organics is below the 5 ppb level.

3) COMPLAINTS

Customer complaints are responded to with a GC and UV analysis. On the basis of these analyses the following observations can be made:

i) Volatile organics may have entered the system through a main break,

ii) Volatile organics may have entered the system through a cross connection,

iii) Volatile organics could already have been present in the treated water when it left the plant.

In all the above cases the flushing of hydrants may be carried out if treated water in the hydrant has higher GC or UV absorbance readings than treated water leaving the plant.

4) SPILL

Spill events are common occurrences at the Rossdale Water Treatment Plant. A summary of spills prepared by the laboratory staff at the Edmonton water treatment plants listed a total of 59 events over the period of three years from 1984 to 1986. During the occurrence of a spill, the frequency of monitoring of the raw water is increased and GC analyses are carried out round the clock.

5) BY-PRODUCTS OF DISINFECTANTS

It was first demonstrated by Rock in 1974 that the chlorination of naturally coloured waters resulted in the formation of chloroform, a suspected carcinogen[5]. GC is routinely used to monitor levels of chloroform in treated water. When the level of chloroform exceeds 5 ppb, the dosages and concentrations of ammonia, chlorine, the level of humic substances and the temperature of raw water are checked, as all of them contribute to the formation of chloroform.

During the 1986 spring runoff, when the level of humic substances in raw water was high, GC analysis indicated the presence of volatile organics which later were identified as three aldehydes. The presence of these peaks coincided inevitably with consumer complaints. It was realized that a high dosage of chlorine dioxide during the runoff produced aldehydes as by-products which, at very low ppb levels, produced an objectionable taste and odour problem in treated water.

ACKNOWLEDGEMENTS

This study was supported by the City of Edmonton. I express my appreciation to Laboratory Staff for routine UV work and to Pearl Poon for preparation of standards and recording data.

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THE MAR-LAR PROCEDURE: REDUCING THE NEED FOR CONFINED SPACE ENTRY WHEN DOING SEWER MAIN VIDEO INSPECTIONS USING A SEWER FLUSHING UNIT AND THE MANHOLE HELPER

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In the past ten years, the potential risk to a sewer serviceman who must enter a manhole or catch basin to guide a pressure hose into a lateral for line cleaning or video inspection has increased to such that most Government regulatory bodies have introduced regulations and procedures designed to reduce the risk factor.

While we understand the need for such regulations, the cost burden involved in the implementation of the regulations is such that in some cases, the regulations are only being partially adhered to or ignored altogether.

If an employer supplies his workers with the equipment and training needed to undertake a Confined Space Entry, it has been found that the employee will usually short cut the procedure to save time and effort, with little or no regard for his own safety.

In addition to the inherent problems of over-seeing workers to ensure proper job safety procedure, the employer has also been delegated the un-enviable task of informing the consumer that costs must rise to pay for high safety standards.

How can we as responsible employers, minimize the risk to our employees, maximize job productivity, reduce job costs, and protect expensive equipment?

The advantages of C.C.T.V. inspections on storm and sanitary sewer systems have become increasingly apparent to both the private and public sectors over the past few years.

Cameras have been developed to the hi-tech level, incorporating concepts such as remote focus and iris facilities, built in radio sound for precise location underground, posiswivel slip-ring connectors for ease of negotiating 90° bends, high intensity, low power lights which are built-in to the camera casing, high quality video monitors, monochrome reproduction, video computer for text insertion on video screen, and solid state image sensors for reliability and durability.

Simplicity in operation of the camera systems has been maintained. In most cases, the camera is simply pushed down the line to be inspected via a semi-rigid, flexiprobe type rod or a type of sewer snake. The procedure is straight forward, relatively lacking in complications with little or no risk to the worker when he is undertaking inspections from a clean-out access.

The scenario changes dramatically when inspections are done on sewer lines which are accessed through manholes or catch basins. Pushing a camera through lines over 150 feet from a manhole access, involves dealing with a completely new and more detailed job procedure to ensure protection of both equipment and workers.

Inspection procedures have evolved based on the technics used for the smaller, more accessible sewer lines. Variations of accepted procedures range from the simple push method to the rather drawn out process of stringing. Crews are required to undertake Confined Space Entry procedures to ensure safe conditions when working inside the manhole or catch basin.

The savings realized by C.C.T.V. inspections are being eroded by high procedure costs.

Crews are being exposed to unnecessary risk by procedures which require them to enter a manhole or catch basin to undertake video inspections.

Expensive equipment is not being protected in an efficient manner.

Clearly, there is a high priority need for a procedure which incorporates safety, simplicity, protection for men and equipment, and is economical.

The Mar-Lar Procedure satisfies all those needs.

It has been demonstrated repeatedly, that unsupervised or poorly supervised crews will short-cut on recommended safety procedures due to a number of reasons, not the least of which is direct or indirect pressure from employer or supervisor to increase job productivity and reduce job costs. Another reason workers have expressed for failing to follow proper safe job procedures is that the procedures are complicated, time consuming and require more men power to set up for an entry than required to do the actual job. Although their arguments are short sighted when considering the potential job risks, the idea of a simpler, safe procedure is undeniably more attractive in relation to safety factor, job costs and maximizing job production.

Much has been done to make entries into manholes and catch basins less risky for the worker. Most, if not all Provinces have developed entry procedures, that, when followed, effectively protect the workers undertaking an entry.

However, little can be found on keeping the worker out in the first place. It seems to have become a prime example of the old saying, "can't see the forest for the trees". It appears in most cases, an entry into the manhole or catch basin is assumed when conducting video inspections. With the Mar-Lar Procedure, an entry would become an exception instead of the rule.

Confined Space Entry — Sewer Entry Guidelines as prescribed by Alberta Occupational Health and Safety Act and Regulations, published in booklet form in August 1986 for the purpose of accident prevention, is extremely helpful in developing job procedures when an entry is required.

With the Mar-Lar Procedure, the need to enter a manhole or catch basin for the purpose of video inspection of the sewer line is effectively reduced.

The Mar-Lar Procedure eliminates the stringing method, the push method, reduces risk to the worker, protects expensive equipment and reduces the costs related to video inspections.

The Mar-Lar Procedure is simple, quick and cost effective.

Reducing the need for Confined Space Entry when doing sewer video inspections, utilizing a sewer flushing unit and the Manhole Helper: The Mar-Lar Procedure.

The Mar-Lar Procedure for video inspection of sewer lines utilizes a skid mounted video camera with lead chain, return cable, a sewer flushing hose with leader, a jet nozzle and a Manhole Helper Set.

The pressure hose is lowered into the manhole, down line of the line to be inspected.

The Manhole Helper brace and roller is lowered into the down line manhole and the extension poles are added to bring the roller to the depth of the sewer line. Brace, roller and poles are secured.

The pressure hose is jetted up the sewer line, propelled by water forced under pressure through the jet nozzle, to the up line manhole.

The pressure hose is brought to street level by snaring the jet nozzle with a shepherd hook.

The video camera and skid are secured to the jet nozzle via the lead chain, using a slip joint end, and lowered from the surface into the manhole.

A manhole helper is also secured in this manhole.

The pressure hose is pulled back through the line, pulling the camera and skid.

Two-way radio communication is maintained between the video monitoring van and the pressure truck operator so the video operator can direct the speed of the pull.

When the camera has been pulled to the manhole, the crew can manipulate the bottom roller of the Manhole Helper so as to bring the camera around it without damage and the hose and camera are brought out of the manhole for disconnection.

The return cable is winched back through the manhole and line and the pull is complete.

The Manhole Helpers are removed from the manholes and the manhole lids are replaced.

The inspection crew moves on to the next pull site.

In most cases, the video inspection crew members are NOT required to enter the manholes with the Mar-Lar Procedure. "Stringing" of the lines for video inspection is NOT required. Flushing of the sewer lines prior to video inspection to ensure clear passage for the camera need not be a separate undertaking. This can be done with a minimum of delay to the camera crew when the pressure hose is jetted up the line for camera connection. Optimum job productivity can be realized when crews are not required to set up for three Confined Space Entries for one video pull.

Safety factor on the job increases substantially when the worker is not required to make a manhole entry.

Physical Hazards of a Manhole

Any or all of the following physical hazards may be present in a manhole, creating potential dangers to the workers.

1. Broken access rungs.
2. Cracked walls.
3. Deep or fast moving effluent.
4. Disease-causing bacteria and viruses.
5. Harmful substances (gasoline, oil, paint, solvents).
6. Oxygen deficiency.
7. Toxic/Explosive gases.
8. Debris (rocks, glass, wood chunks).

Oxygen Deficiency

Chemical action may result in oxygen being used up. An example is the rotting of organic matter, such as sewage, by the action of bacteria. A combination of this and other processes can lead to a reduction of the total oxygen content of the air in the sewer to below the minimum requirement for carrying out work. In Alberta, the minimum oxygen requirement is 18 kilopascals partial pressure (18% at sea level). Below this level, atmosphere supplying respiratory protective equipment (breathing apparatus) is required.

Hydrogen Sulphide (H_2S)

The deadliest gas commonly found in sewers, H_2S can be created by the decomposition of organic matter. Concentrations as low as 1 ppm have a characteristic smell of rotten eggs. NOTE: At higher concentrations the odour of H_2S may not be detected, since the gas affects the sense of smell. Concentrations of over 700 ppm (0.07%) will cause unconsciousness in a few seconds. If the victim is not immediately removed to fresh air, death will follow quickly.

Carbon Monoxide

This is a colorless, odorless and deadly gas. It is the product of incomplete combustion, and the most common source is the exhaust of gasoline engines. If such engines must be operated near an open sewer, precautions must be taken to ensure that the exhaust gases are directed away from the opening. Overexposure may result in a worker experiencing ringing in the ears, nausea, headaches and sleepiness. Unconsciousness and death will follow unless immediate rescue is taken.

Methane

The main danger associated with methane is explosion, due to its flammable nature. It may be released from a leak in a gas line, but can also be a by-product of a backed-up or sluggish sewer. This gas may displace oxygen to below the point necessary to maintain life, acting as an asphyxiant.

Gasoline Vapors

Gasoline is sometimes founds in both sanitary and storm sewers. This may be the result of leaking underground storage tanks, the illegal disposal of gasoline into the sewer system by the public, or by workers trying to clean up a gasoline spill. Gasoline vapors, even in small amounts, can be a fire and explosion hazard.

Statistics

Worker #1	Worker #2	Worker #3	Worker #4
Height: 71"	Height: 71"	Height: 74"	Height: 71"
Weight: 175 lbs.	Weight: 180 lbs.	Weight: 185 lbs.	Weight: 240 lbs.
Shoulders: 23"	Shoulders: 22"	Shoulders: 21"	Shoulders: 24"
Chest: 41"	Chest: 38"	Chest: 36"	Chest: 46"
Hips: 41.5"	Hips: 39"	Hips: 39"	Hips: 46"

Overall Average

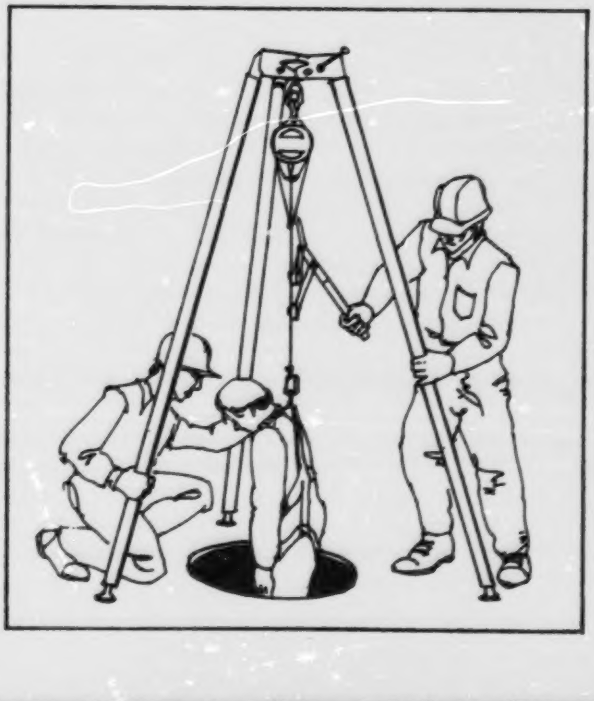
Height: 71.75"
Weight: 195 lbs.
Shoulders: 22.5"
Chest: 40.25"
Hips: 41.38"

The average manhole access in the City of Calgary is approximately 22" across at the lip of the manhole.

An average ventilation hose is approximately 8" across.

An average water pressure hose is 1.25", outside diameter.

An average hose guide is 3/4", outside diameter.



"Is Sewer Entry Absolutely Necessary to Complete the Work Required?"

This is the first question asked under Pre-Entry Planning in the Sewer Entry Guidelines pamphlet, published by Alberta Community and Occupational Health. Until the Mar-Lar Procedure for undertaking sewer video inspections, the answer has been yes.

Given the statistics mentioned for sewer servicemen and the average for the men, and the basic equipment needed to set up for a Confined Space Entry for video inspection, there is very little room for an entry into the manhole, let alone for an exit under stress should an emergency occur. Would it not make more sense to avoid an unnecessary entry whenever possible.

If the safety factor is not enough to convince workers and employers of the need to avoid unnecessary entries, consider the cost savings. The following comparison is based on work done with the City of Calgary video inspection crew. Figures used for footage pulled and videoed, labour costs and equipment costs, are estimates and amount of potential savings will depend on methods currently used by individual crews and the cost for labour and equipment in a given local. If a crew is currently using a stringing method, the potential cost savings could conceivably be considerably higher.

COST COMPARISON ANALYSIS

Confined Space Entry on a Video Camera vs

The Manhole Helper on a Video Camera Crew

Confined Space Entry:

600,000 feet videoed per year
average pull length 400 feet
1500 pulls per year
3 confined space entries per pull
4500 entries per year

1 pull takes an estimated 40 minutes to properly set-up for Confined Space Entry (time total is inclusive for entries required on one pull).

For 1500 pulls: 40 times 1500 = 60,000 minutes or 1,000 hours/year

Assuming the cost of labour is: 50/hr for the pressure truck and operator
18/hr video crew supervisor
28/hr for two laborers
96/hr total labour costs

Cost of labour for Confined Space Entry: 1,000 hrs times \$96 = \$96,000.00

In addition to the labour cost factor, the crews are still required to enter the manhole approximately 4500 times.

Manhole Helper:

600,000 feet videoed per year
average pull length 400 feet
1500 pulls per year
No confined space entry is required

1 pull requires 10 minutes total set up time for the Manhole Helper

1500 pulls times 10 minutes = 15,000 minutes or 250 hours

250 hours times $96 per hour = $24,000.00

NO ENTRIES REQUIRED

Potential savings in labour costs alone may be as much as $72,000.00.

The risk to the worker has been dramatically reduced due to the fact he no longer is required to enter the manhole.

Benefits of the Mar-Lar Procedure

1. Effectively reduces the need for Confined Space Entry during a video inspections of sewer lines.

2. Can be easily set up and removed by one man.

3. The Manhole Helper is light weight and easily stored on the pressure truck.

4. The Manhole Helper is simple to use on the job and takes very little time to install and remove from the manhole rim.

5. The Manhole Helper is inexpensive.

6. Once installed in the manhole, the Manhole Helper acts as a deterrent to entry. The worker would have to physically remove the bracing arm to enter the manhole.

7. The Manhole Helper protects equipment by guiding hoses, camera and cable thereby reducing damage due to rough/damaged sewer lines.

8. Job productivity increases due to set-up time saved when Confined Space Entries are reduced.

9. Job risk to the worker is dramatically reduced when inspections can be done without a Confined Space Entry.

10. The worker will choose to use the Manhole Helper and the Mar-Lar Procedure because it will make his job easier.

11. The Manhole Helper is adjustable to most manhole and catch basin lips.

12. The Manhole Helper and the Mar-Lar Procedure can be used with nearly every type of video camera designed for sewer inspection.

13. Training for the procedure takes as little as one day.

14. The Manhole Helper will pay for itself very quickly.

FIGURE 1

Section C-C

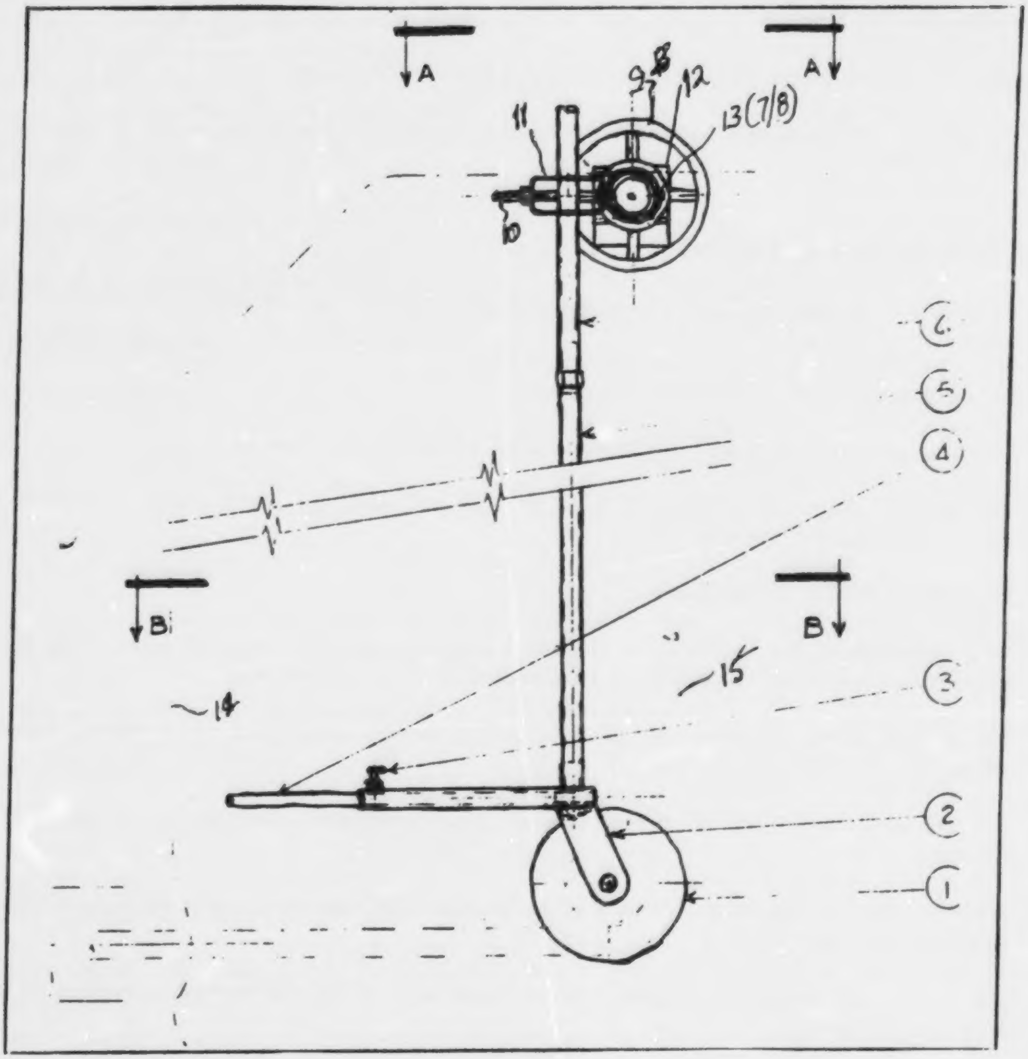

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FIGURE 2

Section B-B

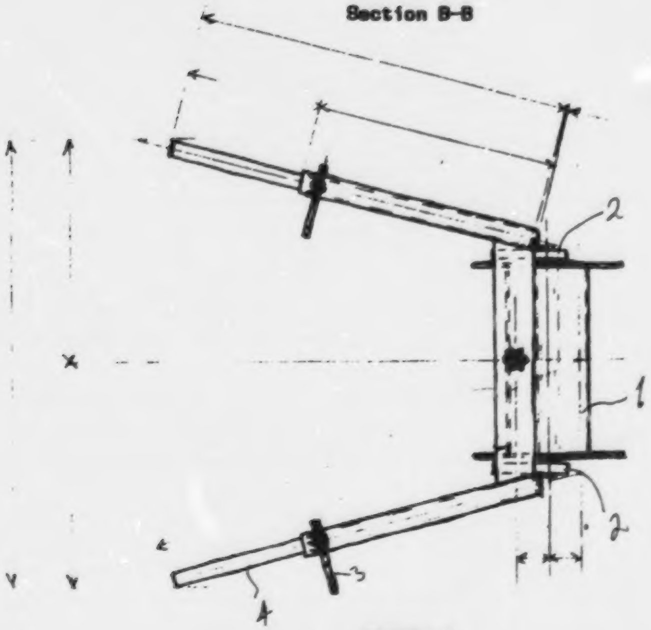
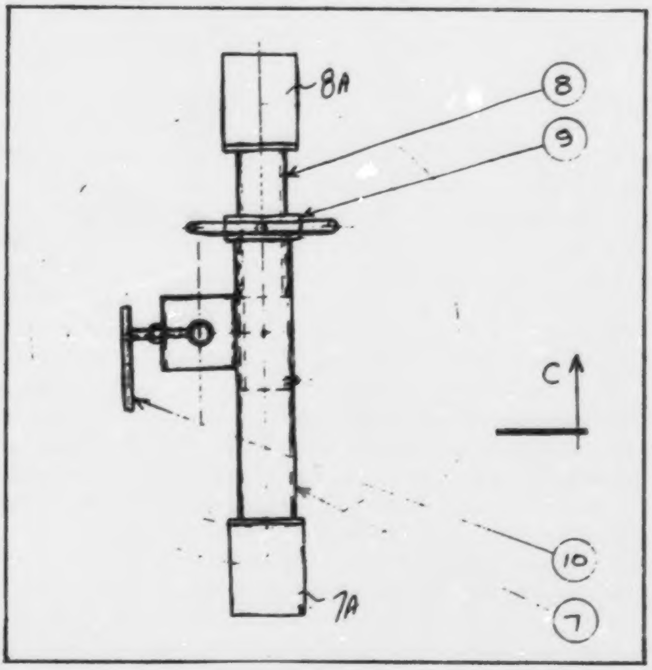


FIGURE 3

Section A-A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CATHODIC PROTECTION FOR BURIED IRON FITTINGS BASED ON MARINE RESEARCH

Jack Taylor
President
Cantii Consultants Inc.

There are a number of factors to be considered in the design of a system for the Cathodic Protection of buried iron fittings. The following are of importance.

Soils

More than 200 soil types have been categorized in the United States. The various categories can be determined by Texture, Colour and Drainage. These physical properties greatly concern the Corrosion Engineer as he is primarily interested in the permeability of Air and Water. Soils with a coarse texture, such as sands and gravels, permit free circulation of air. Corrosion in these categories closely approaches that occurring in the atmosphere. Clays and Silty soils are more generally characterized by aeration and drainage.

The aeration characteristics of the soil are dependent upon particle size and distribution and on the apparent specific gravity. The latter being a measure of the compactness of the soil. Numerous chemical elements are present in soils, but the ones of primary interest in corrosion studies are those which are soluble in water. Analyses can be made for base-forming elements such as sodium, potassium, calcium and magnesium, and for the acid-forming groups such as carbonate, bicarbonate, chloride, nitrate and sulfate. The electrical resistivity of the soils is determined by the nature and concentration of the ions formed by the chemical salts dissolved in the soil moisture. These ions may also help determine the progress of corrosion. However, if the primary product of corrosion is relatively insoluble and deposits as a film on the metal surface, further corrosion may be reduced or completed stifled.

Pure Zinc

The Pioneer efforts to use zinc as an anode in underground systems protecting steel pipe and other ferrous structures go back to the 1930 era. Pure zinc anodes were installed bare in a wide variety of soils with mixed results. 'Walquist' reported some cases in which anodic surface films formed which significantly increased the circuit resistance and reduced the current output. Typical Iron content of these "Pure Zinc" anodes was about 0.008% and often values in excess of 0.01% were encountered. Unfortunately, no record was made of Iron Content in relation to the mixed performance. 'Mudd' observed relatively consistent performance of zinc anodes when installed in soils containing gypsum (hydrated calcium sulfate). As a result he suggested packing gypsum around "Pure Zinc" anodes when installation was to be made in aggressive non-gypsum soils. These early experiments concluded, that the environment around a "Pure Zinc" anode, in a large measure, determined the nature of performance of the anode. Soils containing significantly more dissolved sulfates and chlorides than carbonates, bicarbonates, nitrates and phosphates, are often compatible with "Pure Zinc" anodes. The current practice of packing a prepared backfill consisting of hydrated gypsum, bentonite clay and sodium sulfate around a "Pure Zinc" anode, creates an artificially friendly environment, however, time and availability of water causes the sodium sulfate to leach out and higher resistivity occurs.

Primary problems have always occurred related to the design location and attachment of these relatively large "Pure Zinc" anodes and in recent years continuous "Pure Zinc" ribbons have been used on a variety of underground applications. This latter product has broadened the application for "Pure Zinc" anodes, for it provides small increments of current along the entire length of a cathode. There are reports of considerable success of this type of Cathodic protection where other methods are either impractical or extremely costly.

Sea Water

In the immediate post World War II period, the largest consumer of zinc galvanic protectors in the U.S.A. was the US Navy which, at that time, specified rolled zinc plates - bolted to the stern areas - for a great variety of vessels. This was done in an effort to mitigate corrosion of the steel in the general vicinity of copper base or other alloy propellers which were cathodic to steel. Careful hull surveys often indicated spotty performance of the "Pure Zinc" plates, to the extent that a significant percentage of the plates failed to function anodically. In an effort to determine the reason for this relatively poor performance, the American Zinc Institute (now the Zinc Institute) funded a research program carried out at the International Nickel Company's (now INCO) sea water test facility at Harbor Island N.C. The major finding of this report was that "Pure Zinc" functioned as an anode in sea water when the iron content was maintained at a very low level. A recommendation was made to limit the iron content to below 0.0015% in otherwise "Pure Zinc" to avoid the formation of anodic corrosion product films of very high electrical resistance.

In 1956 the U.S. Government issued a specification (MIL-A-18001A) for "Pure Zinc" anodes in which the iron content in unalloyed high purity zinc was limited to 0.0014% in the finished product. In order to meet this rigid specification, anode producers used special high grade zinc (SHG) slab, 99.99% pure with an iron content of generally less than 0.0008% as their starting stock.

Laboratory and field studies by 'Crennel & Wheeler' showed than an iron content level of more than a few parts per million caused passivation of zinc and that no other common impurities were of importance to anode performance. Their studies showed the addition of 1% aluminum was beneficial and 0.5% aluminum plus 0.5% silicon significantly increased zinc anode's tolerance for iron and also produced a finer grain structure than was obtained with SHG zinc with 0.0014% maximum iron.

'Reichard & Lennox' conducted full size zinc anode field tests up to 15 months on a sea water bulkhead, a T-2 tanker, tugs and a Navy YO class tanker. Their data supported the earlier finding that neither aluminum nor cadmium, when added alone to zinc, eliminated the need for controlling the iron content to very low levels. However, when small amounts of aluminum and cadmium were added to zinc, the tolerance for iron increased substantially.

Subsequently, long term field tests reported by 'Waldron and Peterson' and also by 'Carson', utilizing full size zinc anodes with aluminum plus cadmium as alloying additions confirmed the field exposure data of 'Reichard & Lennox'.

Another paper, by 'Carson, Phillips and Wellington', concerned the effects of the addition of aluminum and iron to zinc anode performance. Their data shows how the potential of zinc shifts in the noble direction during sea water exposure of 60 days. That shift becomes more pronounced as the current or iron content increased. Addition of 0.3% aluminum to the zinc alloy retarded this potential shift. The authors estimated the optimum aluminum content at approximately 0.5%.

The Russian researcher 'Lyublinskii' also confirmed that approximately 0.5 - 0.7% aluminum gives the maximum performance in iron bearing zinc anodes. Further data from 'Vassilev, Petrov & Krasteve' also confirm the effects of iron and aluminum on the long term exposure potential of Zinc anodes. The role of the grain size on Zinc scale is still not fully understood, however, a finer grain size appears to lead to a finer size of harmful inclusions, thus reducing the tendency for zinc oxide nucleation.

The U.S. Government recognized this wealth of research and modified the Zinc Anode Specification [MIL-A-18001F] to include the addition of aluminum and cadmium to zinc anodes for fleet use. This was later modified to [MIL-A-18001H] and also ASTM B-418-80, Type 1. carefully controlling the maximum amounts of Iron, Lead, Copper and Silicon and defining the limits of Aluminum and Cadmium. This latter specification has become the Standard for Marine application worldwide.

Continuing the research of earlier years in 1986 'Foulkes and Hinatsu' of the University of Toronto conducted a series of experiments based on the use of the above alloy applied as a Cap directly to Iron Bolts on a series of conventional Iron Fittings submerged in an aerated synthetic seawater under ambient temperatures. The voltages of the nuts and bolts and the voltage distributions in the iron assemblies were monitored by the use of a high impedance electrometer and a number of areas were scraped bare of the Black protective coating in order to simulate normal field handling. Research was also done on the substitution of the Iron Nuts and Bolts by stainless Steel alternatives. After initial surges in potential all experiments settled to a point of relative stability by the 300hr test period. The tests were confirmed without modification to the experimental set-ups for a period of a little over 1000 hrs.

The University of Toronto conclusions were as follows:

1. Zinc Caps confer excellent Cathodic protection to Ductile Iron mechanical joint fitting and gland ring assemblies.

2. Increasing the number of Zinc Caps marginally increases the degree of Cathodic protection conferred to the Ductile Iron.

3. Increasing the number of Zinc Caps used greatly increases the length of protection.

4. Stainless steel nuts and bolts should be avoided for use with ductile iron assemblies since such nuts and bolts will accelerate the rate of attack of the iron assemblies, especially attack at the flange areas in the vicinity of the stainless steel components.

Conclusions

It is not difficult to deduce that the lines of argument based on the research done in both soil and sea-water applications for Zinc Anodes lead to the facts that the mineral content of the soil and the iron content in the Zinc are very important for the performance of the anodes.

The work done by the Marine researchers had the advantage of being readily accessible at all times. It is not quite that simple to test the long term performance of buried anodes. However the recent research by the University of Toronto and the stability of performance of the Zinc Caps, after the initial settling down, also supports the theory that Zinc with carefully controlled amounts of iron and small amounts of both Cadmium and Aluminum has almost guaranteed performance as an Anode for its design life.

The metal content of the "Protecto-Caps" and "Protecto-Collars" takes advantage of this research by producing a virtually "foolproof" product that is simple to attach under almost all conditions - even submerged - to provide long term Cathodic protection on Iron fittings.

CREDITS

Zinc Institute and University of Toronto

THE GENERAL ELECTRIC GENIUS I/O AND INCREASING RELIABILITY IN WATER AND SEWAGE CONTROL

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Introduction

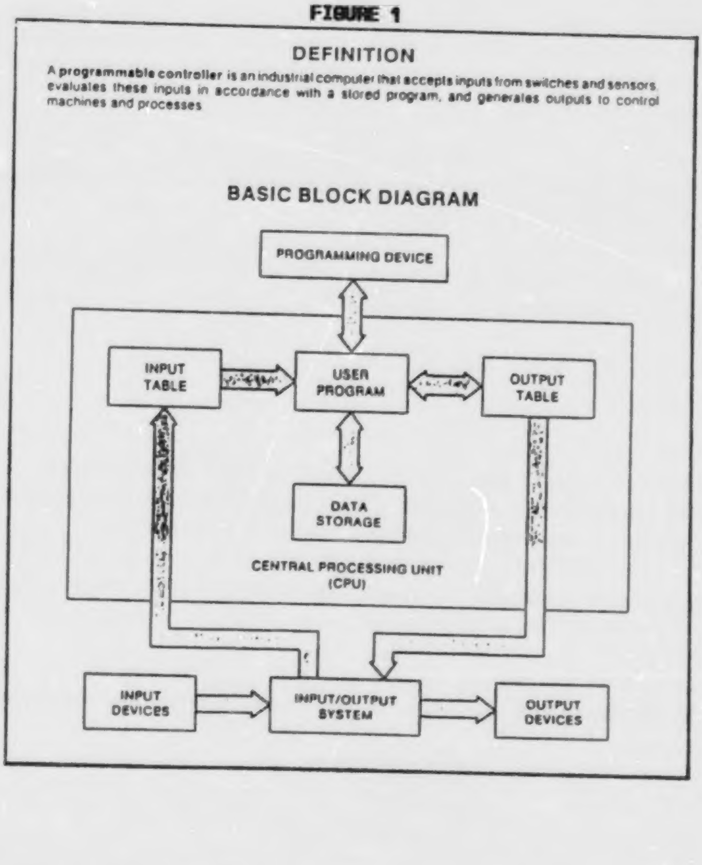
The application of the PLC (Programmable Logic Controllers) to municipal water distribution and treatment facilities is only one of many possible uses for these powerful and versatile industrial computers. A PLC can easily be applied to remotely controlled motor start-ups and shut-downs, level control, metering, alarming, security and more.

There may be no need to explain what a PLC is, but a brief description may be of benefit. The National Electric Manufacturer's Association (U.S) defines a PLC as:

> "Programmable Controller - a digital electronic apparatus with a programmable memory for storing instructions to implement specific functions."

A more basic explanation is to think of a PLC as a "black box full of relays, timers and counters". These relays, timers and counters are used, as required, to perform specific control tasks. In their simplest form, all PLC's have three major components: Input, output and a processor. (Figure 1)

FIGURE 1



DEFINITION

A programmable controller is an industrial computer that accepts inputs from switches and sensors, evaluates these inputs in accordance with a stored program, and generates outputs to control machines and processes.

BASIC BLOCK DIAGRAM
PROGRAMMING DEVICE
INPUT TABLE
USER PROGRAM
OUTPUT TABLE
DATA STORAGE
CENTRAL PROCESSING UNIT (CPU)
INPUT DEVICES
INPUT/OUTPUT SYSTEM
OUTPUT DEVICES

Therefore, by definition alone, all PLC's seem the same. They all can sense inputs, manipulate them according to a particular program, and control outputs.

But all PLC's are not the same. Manufacturers provide various models and features to best allow for specific control applications. Features such as floating point mathematics and communications are definitely not required for all control schemes. This is why even within a specific PLC model there will usually be a range of option modules for interface to the many types of field instrumentation.

The PLC is usually programmable in Boolean logic or relay ladder logic. While the advantages of other programming languages are many, and their use is slowly and steadily increasing, the ladder diagram is still popular and prevalent because of its strong visual presentation. It is easy to see how the logic solutions happen. Many PLC manufacturers offer ASCII-BASIC modules to allow users to create specific application programs.

Before concluding this basic explanation of PLC's, the PC should be mentioned. The abbreviation PC is common for personal computers, especially those manufactured by IBM. The PC is rapidly becoming the both programming device for the PLC and the operator interface. The PC is capable of loading, monitoring and editing the program plus displaying the required operational graphics and alarms. A PLC that operates easily with a PC is definitely a benefit for the user, both in cost and versatility.

If PLC's seemingly offer the same features, abilities and benefits from one manufacturer to another, then how can you make a decision? Especially if all submissions seem to meet your specifications.

A purchase made solely on price, many encounter problems such as a lack of supplier support services or basic ability "to do the job". The lower cost PLC may require expensive [and extensive] input and output signal conditioning, lack speed or memory, or be very difficult to program and document.

A prospective user must therefore look for "features" that make operating and maintaining the PLC easier [more "user friendly"].

Such a feature is the major advancement to PLC I/O [Input/Output] control, announced in 1985, that remains unique and unsurpassed in its benefits.

This advancement is the GENIUS I/O System.

What is GENIUS I/O?

Imagine intelligence in discrete and analog input/output so advanced that it not only takes its own pulse, but also automatically pinpoints faults outside the programmable controller. All the way to individual circuits, hard-contact sensor wiring and actuators.

Imagine an I/O system that requires no racks. No separate power supplies. No receivers. No transmitters. No special cables or connectors.

Imagine an I/O system that allows you to mix and match inputs and outputs on a single block in any combination you choose. Without the need for separate input and output modules.

Imagine a new distributed I/O concept that can dramatically reduce your installation costs. And lets you put I/O in places where you never could before.

That's the GENIUS I/O system. It will change forever the way you use programmable controllers.

The GENIUS I/O, a system of inherently distributed inputs and outputs, represents a complete re-thinking of the role of I/O in industrial control. The GENIUS I/O system is made possible through two key technological innovations.

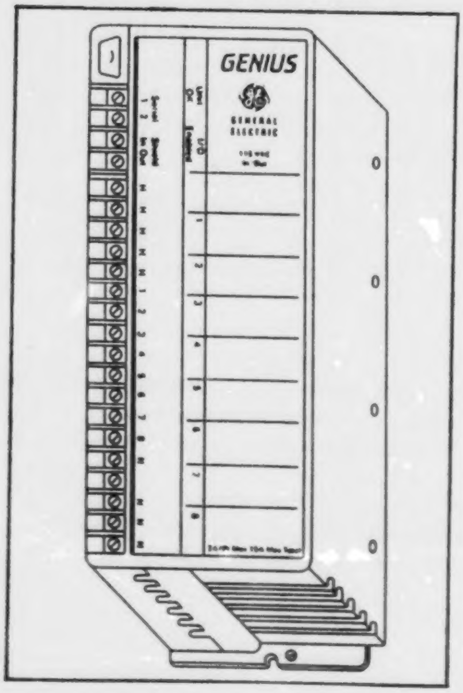
- Smart Switch: A device with built-in current and voltage sensors required for the extensive diagnostics available with GENIUS I/O. The smart switch allows detection of faults not only within the programmable controller I/O system, but also faults in the coils and other actuator devices under the control of the programmable controller, as well as the signal path from pushbuttons and other input devices. No other technology provides this level of fault detection.

- Communications Controller: A token bus local area network controller that allows GENIUS I/O devices to communicate over a single shielded twisted wire pair, rather than via bundles of point-to-point wires required in conventional systems.

The GENIUS I/O system consists of the following four elements:

- I/O Block: A microprocessor-based, configurable, ruggedized solid state device to which field I/O devices are attached. The block can be mounted virtually anywhere. No separate rack or power supply is required. Field wiring is attached to a terminal block section which separates from the removable electronics package. Thus, field wiring need not be disturbed to service the electronics. Due to the microprocessor and intelligent switching, inputs and outputs may be mixed arbitrarily on blocks. There are no dip switches nor replaceable fuses. (Figure 2)

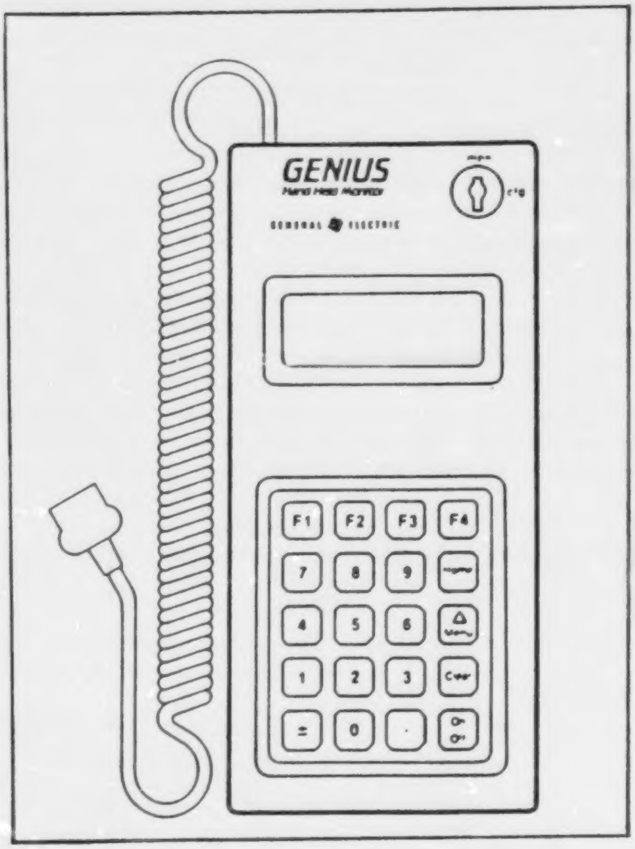
FIGURE 2
Typical GENIUS I/O Block



- Hand Held Monitor (HHM): A portable diagnostic and configuration tool used for addressing, trouble-shooting, monitoring, scaling and configuring the I/O Blocks. The HHM plugs directly into any block or into the programmable controller. It has an alphanumeric LCD display with microprocessor-driven prompts available in several languages. A key feature of the HHM is its ability to manually perform functions and force discrete and analog I/O, whether or not there is a PLC connected to the system. This greatly facilitates system check-out prior to full-scale operation. (Figure 3)

FIGURE 3

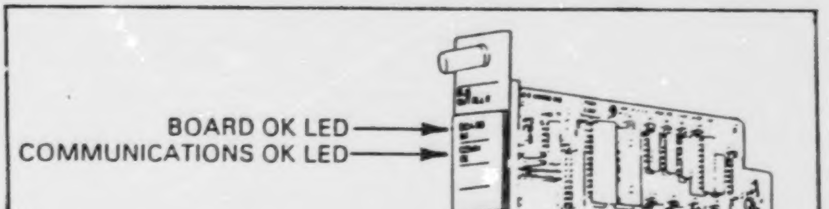
Hand Held Monitor



- Bus Controller: The interface between the GENIUS I/O system and the PLC. The Bus Controller accommodates up to 30 I/O Blocks or Hand Held Monitors at distances up to 2000. Multiple Bus Controllers can exist. The Bus Controller also provides diagnostic fault reporting to the PLC. (Figure 4)

FIGURE 4

Bus Controller



- GENIUS I/O Bus: a 150 kbit/sec serial token passing bus providing communications between the Bus Controller, Hand Held Monitors, and I/O Blocks. It has high noise immunity and its operation is not affected by any block attachment, removal or failure. Each data bit is triply encoded for data integrity; error detection is further improved via cyclical redundancy check (CRC). Bus errors are reported automatically.

HOW DOES GENIUS WORK?

There are three major operation features that enable GENIUS to provide the diagnostics that will benefit maintenance. These are:

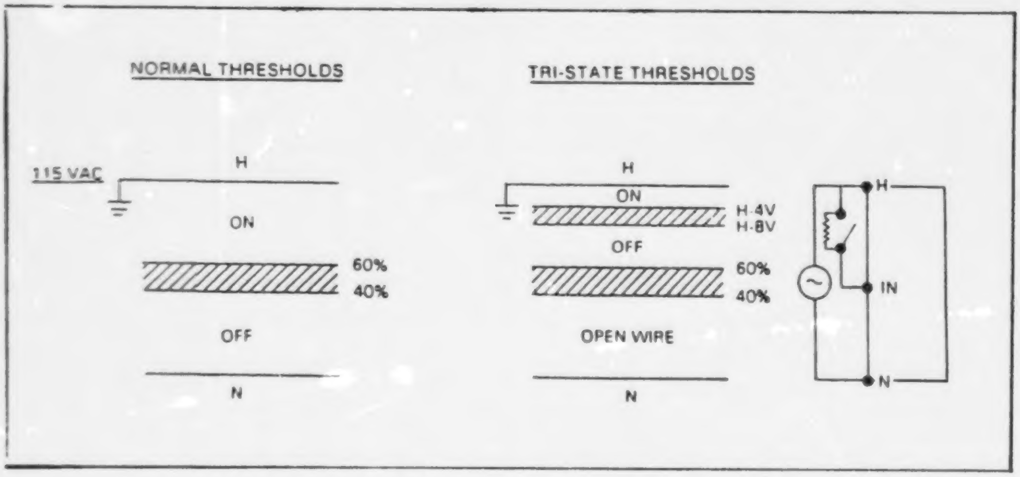
Tri-State Inputs
Output Currency Sensing
Output Pulse Test

Tri-State Inputs

For discrete inputs, Open Wire detection is provided by configuring an input for the "Tri-State" diagnostic. Tri-State inputs are used in conjunction with devices which do not contain active electronic circuitry. Such circuitry is referred to as "dry contact" including such mechanical devices as pushbuttons, relay contacts, limit switches, auxiliary motor starter contacts, and toggle switches. (Figure 5)

FIGURE 5

Tri-State Input Test



To distinguish between ON, OFF, and Open Wire conditions, an external preload resistor is placed as close as practical to the actual dry contacts (such as across the field device terminals). If the GENIUS input senses a low impedance, the field device is closed (ON state), a nominal impedance indicates the field device is open (OFF state), and a high impedance indicates the circuit has opened, resulting in an Open Wire diagnostic. When malfunctions are detected, the CPU can be notified and the input will be set to the OFF state.

Output Current Sensing Levels

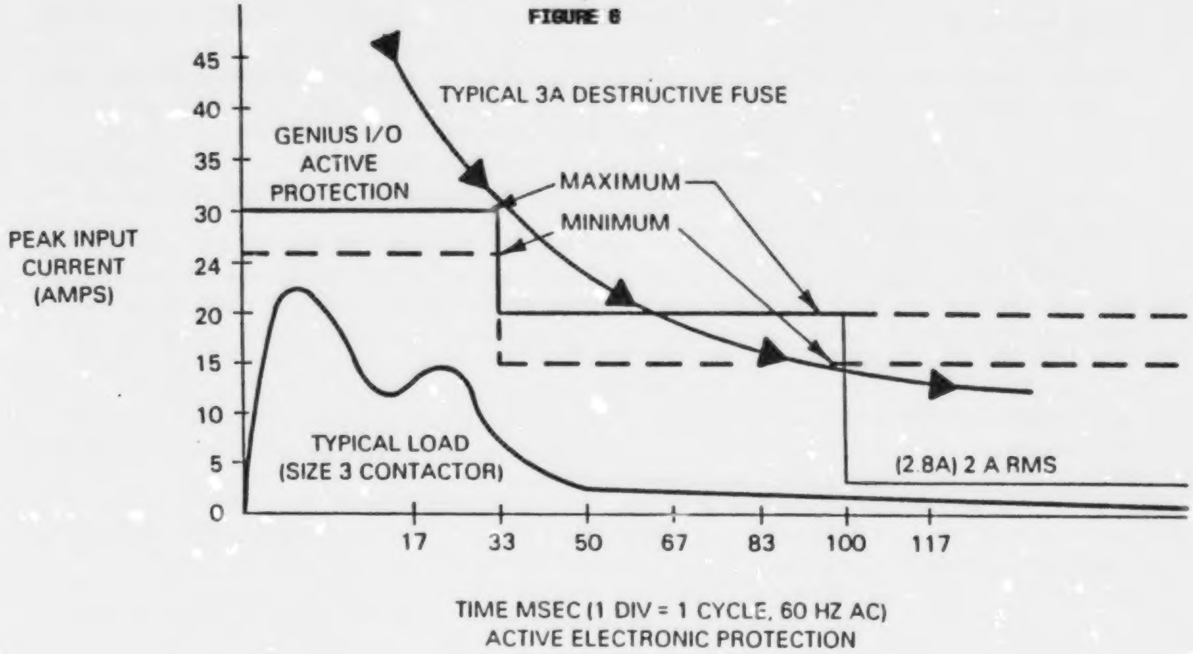
Discrete output blocks sense current flow when each circuit is turned ON. Separate sensors enable the GENIUS I/O block to evaluate each circuit independently, even if multiple outputs are energized at the same instant. (Figure 6)

The lowest level verified determines if a load is connected. Current flow below this level is considered as no load present.

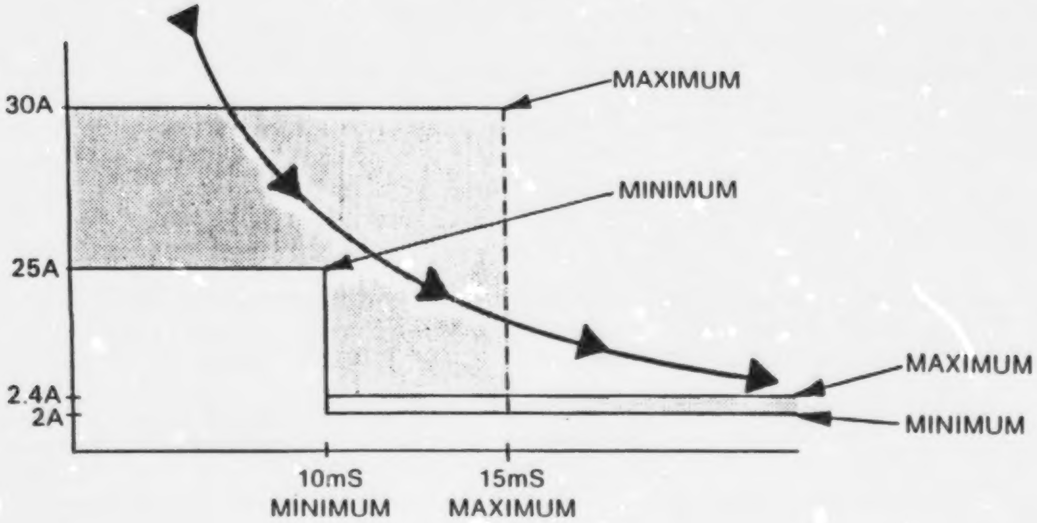
A middle level (2.0 amp value) represents the steady state maximum current flow from each circuit. If the output current exceeds 1.0 amps for 100 ms on AC outputs or 10 ms for DC outputs, the output will turn itself OFF to protect the load, field wiring, switching device, and power source. An Overload alarm will be set.

The highest level provides short circuit protection. Circuits so configured are subsequently protected by a 20 amp short circuit level and temperature sensors at the switching device. Transient currents of 20 amps or greater will cause the switching device to turn itself off. A short circuit alarm will be set.

FIGURE 6



AC Output Protection



DC Output Protection

Output Pulse Test

Outputs can be configured to check for the ability to change state by performing a Pulse Test. A user command in the CPU indicates when and how often the output circuit test is performed.

The pulse test is useful for outputs that remain in one state (ON or OFF) for long time periods. If outputs are OFF, they will be turned ON for approximately 1 ms; if they are ON, they will be turned OFF for approximately 1 ms.

The pulse test generally does not provide sufficient energy to activate mechanical devices such as motor starters, relays or solenoid valves. However, by monitoring current flow, circuit continuity (switching device, power source, wiring, fuses, circuit breakers, or output device coil) is evaluated by the block.

If the output was OFF, no current flow when the output is pulsed indicates an open circuit or failed switch. Similarly, continued current flow when ON outputs are pulsed OFF indicates a shorted switch (both malfunctions are reported to the CPU).

These are not the only diagnostics available (see Figure 7) and combining the full features of the GENIUS with the PLC will provide for increased system availability.

FIGURE 7

GENIUS Diagnostic

<table>
<tr><th colspan="2">SYSTEM LEVEL</th></tr>
<tr><td colspan="2">Addition of I/O Block
Loss of I/O Block
Address Conflict
Bus Error
Bus Controller OK</td></tr>
<tr><th colspan="2">DISCRETE I/O</th></tr>
<tr><th>INPUTS</th><th>OUTPUTS</th></tr>
<tr><td>Open wire
Overtemperature
*Loss of power
*Isolated I/O ONLY.</td><td>Failed switch
Load not present
Overload
Short circuit
Overtemperature
*Loss of power</td></tr>
<tr><th colspan="2">ANALOG I/O</th></tr>
<tr><th>INPUTS</th><th>OUTPUTS</th></tr>
<tr><td>High alarm
Low alarm
Under range
Over range
**Open wire
**1-5 volts DC (4-20 mA) range ONLY.</td><td>Under range
Over range</td></tr>
</table>

FEATURES AND BENEFITS

GENIUS I/O Features

All GENIUS I/O features are standard within each discrete or analog block, and there are no model number variations to complicate selection or stocking. Each system can be tailored to match user requirements; you simply select which GENIUS I/O features you wish to implement.

Specific GENIUS I/O features and attributes include:

- User defined mixing of inputs and outputs on discrete (ON/OFF) blocks.
- Rapid, accurate reporting of failures; immediate identification of faults as either internal or external to the programmable controller. In many cases, the GENIUS I/O subsystem defines exactly what failed; power source, wiring, field device, block communications, or internal power switching device.
- Addition or deletion of I/O blocks from the subsystem without affecting the operation of other existing blocks.
- Mixing of high-level analog signals on one I/O block (+-10 volts DC, +-5 volts DC, 0-10 volts DC, 0-5 volts DC, 1-5 volts DC, or 4-20 mA).
- Electronic fusing to protect power switching devices and user wiring. No destructive fuses have to be mechanically replaced when overcurrent conditions occur. Electronic fuses can be reset from the CPU, or with the Hand Held Monitor.
- Detection of open external wires for dry contact inputs (push buttons, relay contacts, and limit switches), discrete outputs (motor starters, solenoid valves, and relay coils), and some analog I/O signals (4/20 mA or 1-5 volts DC). Monitoring can be performed before the device is used, locating faults before the control system is affected.
- Discrimination between short circuits and overloads during the operation of discrete outputs.
- Establishment of linear conversions from analog values to engineering units, such that all data at the CPU is in user-defined units.
- Establishment of output status to be used as a default if the CPU or communications are interrupted.
- Detection of I/O Block additions/deletions by maintenance personnel at remote locations. The system can be programmed to automatically respond to these changes and perform the appropriate action.
- Establishment of high-level and low-level alarms on analog inputs to automatically warn the CPU of undesired analog levels.
- Detect over-range and under-range analog signals and automatically report these malfunctions to the CPU.

- Establish different filter delays on inputs: short filter delays (5-10 ms) for fast system response in controlled low-noise environment or longer filter delays (20-1000 ms) to reject electrical noise in harsh environments.

- Perform analog scaling without using trim potentiometers.

- Detect high temperature levels within the I/O block caused by high ambient temperature and/or overdriving the output switching device.

All GENIUS I/O features are easily user-applied. I/O block defaults are factory set for user-selectable options, and if these defaults are acceptable, only minimum user set-up is required. To enhance GENIUS I/O flexibility, all I/O block selections are performed electronically; there are no mechanical dip switches, jumpers, or address pins to set.

Application Benefits

The potential benefits to be derived from the GENIUS I/O system are of two major types:

- Reduction in installation and operating costs

- Productivity increases

Benefits in the latter category may also be perceived in terms of reduced operating costs, but have a clear impact on reduced downtime for equipment and more efficient use of personnel. Several analyses define these benefits. The results are summarized below:

- Diagnostics: The GENIUS I/O system provides advanced diagnostic capability. This includes a variety of block failure modes, bus failures and failures within the Bus Controller. Of greater significance, however, is the diagnostic power for the attached I/O devices. The system can detect open circuits, short circuits, overloads and a variety of other malfunctions which are beyond the power of conventional programmable controllers to detect. Many faults may be detected before they cause a malfunction in equipment. The GENIUS I/O can detect the integrity of a control circuit before the circuit must actually be energized. This is made possible by periodic "pulse-testing" under microprocessor control within a GENIUS I/O Block.

  An additional benefit derived from using a GENIUS I/O system is the power to specifically isolate and identify faults to the circuit level, and thereby dispatch the right maintenance person with the right tools and parts.

- Inherently Distributed I/O: GENIUS I/O Blocks are mounted at the point of control. The distribution panels, ducts, and auxiliary power supplies associated with most "remote I/O" systems are largely avoided.

- Reduced Installation Costs: for wiring, terminal blocks, conduit, and junction boxes. The reduced installation cost benefits derive from the simpler wiring and reduction in custom panels and ducting — saving in both materials and labour. The entire I/O subsystem can be tested for possible malfunctions before you begin your program.

- Circuit Protection: GENIUS I/O Blocks provide electronic fusing, and overload protection. This is enabled by GE's unique smart switch technology. Smart switches respond within 5 microseconds to a detected short circuit fault, providing far more effective protection than thermal fuses. Short circuits and overloads may be reset from the programmable controller or Hand Held Monitor. Available productivity benefits are reduced maintenance time and better protection of equipment.

- Configuration Flexibility: GENIUS I/O Blocks are defined by general I/O category (e.g. 8 Circuit 115 VAC). Within this category, any of the circuits may be configured by the Hand Held Monitor to be either an input or an output. A single 8-circuit block, for example, is field-configurable to any of 256 distinct combinations of inputs and outputs. Moreover, the system has been designed to minimize and standardize all associated hardware for mounting and cabling. The cost benefit is a potential 60% reduction in parts and spares inventory.

- Intelligence: Many normally hard-wired features of I/O systems are programmable in a GENIUS I/O system. The embedded microprocessor and smart switches allow filter time constants, default values, and other characteristics to be field-configured and customized for the attached devices. The cost and productivity benefits are reduction in initial custom engineering and improved use of equipment.

Beyond these benefits, the flexibility, power and intelligence built into the GENIUS I/O system can remove many of the conceptual obstacles facing production and equipment-design engineers; they are the mechanisms that will allow GENIUS I/O systems to monitor, control and diagnose the next generation of devices in tomorrow's applications.

SYSTEM DESIGN

The objectives of an automatic control system, for either freshwater or effluent control, will typically contain the following design parameters:

1. Automatic control to permit unattended operation.

2. Distributed control using telco facilities or radio communication.

3. Ability to indicate and record tank levels and flow plus display pump status and tank level alarms.

4. Be able to generate and display alarms.

5. Suitable for pump station environment installation.

6. Provide for general public safety.

7. Provide for a level of facility security.

8. Be easy to program, modify, troubleshoot and repair.

If these design parameters are applied to a PLC with GENIUS I/O then several benefits can be demonstrated.

The PLC and GENIUS I/O can obviously fulfill the requirements of the first five (5) objectives. These are normally basic PLC requirements.

The objective of general public safety applies to the unexpected. The loss of power to one GENIUS Block will not affect other blocks, need not stop the PLC and will be automatically reported. GENIUS Outputs can be selected to fail-on or fail-off or hold last state in case of a PLC failure. Knowing what to expect provides for a more orderly shut-down or emergency procedure.

Outputs, which are only used in emergency situations can have their integrity verified with GENIUS pulse diagnostics to locate many failures before the equipment is called into use.

GENIUS I/O can offer more than "door alarms" security. GENIUS diagnostics can report if wiring is tampered with using open wire, no load and short circuit alarms. While not as critical as some buildings, unattended municipal facilities are always targets for vandalism or even sabotage.

The benefits of GENIUS in troubleshooting and repair have been previously highlighted, but consider the following possibility:

At a treatment plant it is necessary to locate motor control in an area where hazardous gases accumulate. Equipment failure troubleshooting requires extensive purging and protective gear for maintenance staff. Often the problem is a blown fuse or tripped overload that is simply reset to clear the fault. With GENIUS such faults could be located and cleared without entering the area.

This can also apply to equipment located in restricted access areas.

Plus, equipment can be controlled (on-off) locally or, in the case of analogs, be positioned to a desired state, in English, away from the PLC.

RELIABILITY

The matter of reliability with PLC's often appears as a negative factor to their implementation.

This is no longer valid as a deterrent to applying PLC's.

The manufacturing and assembly practices coupled with component and assembly testing produces units with failure rates less than conventional relay systems, especially when repair times are considered.

Reliability is directly related to availability, therefore the quicker a system can be repaired, the more reliable it becomes.

In fact, reliability is influenced more by the following factors then the PLC itself. These are:

- Installation techniques
- Power and grounding
- Ambient environment
- Programming techniques (software)
- Training

To further increase GENIUS reliability beyond the user selected diagnostics, sensors are used to verify the states of the Smart Switches. When an output is told turn ON, the sensors ensure that the switch has closed and placed voltage at its output terminal. Outputs told to turn OFF are monitored to ensure that the switch has opened and the voltage at its output terminal has been removed. Failure of either test can result in a SWITCH FAILURE diagnostic message sent to the CPU. This diagnostic message includes identification of the failed circuit.

It is possible for the switch to fail closed or shorted. Under these conditions, current flow cannot be interrupted even if the output state is forced OFF; action external to the I/O Block must be taken. The SWITCH FAILURE message can be used to alert personnel or cause alternate logic to be activated, possibly shutting off power to the block, I/O section, or process.

Note

Monitoring is automatic and requires no configuration or selection by the user. Every discrete I/O circuit configured for an output will have monitoring continuously performed.

If you take the best equipment, install it with proper practices, and apply the benefits of GENIUS, then reliability will be further increased through increased system availability.

CONCLUSION

Obviously, not all applications will suit GENIUS. General Electric has the software and hardware to fully provide for all desired control requirements, regardless of system size.

The GENIUS is showing the future of distributing I/O to the point of control with the ability to individually configure this I/O to local requirements.

Possibly the future will see a universal I/O block that can accept digital and analog signals, interface to any PLC or PC, plus do some control logic within the block.

EFFECT OF BIO-AUGMENTATION PROCESS ON SLUDGE PRODUCTION IN ACTIVATED SLUDGE REACTORS FOR MUNICIPAL WASTEWATER TREATMENT

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INTRODUCTION

Most wastewater treatment facilities utilize biological processes for the treatment and stabilization of organic matter. These processes depend on the action of a heterogeneous population of microorganisms. With an adequate food supply, microorganisms will grow, reproduce and manufacture the enzymes necessary for the stabilization of the waste material. Historically these microorganisms have been naturally occurring species which can adapt to the particular environment of the treatment unit.

Those microorganisms which are primarily responsible for the degradation of organic wastes are saprophytic (organisms that live on dead or decaying organic matter) bacteria. However, too often due to environmental factors, these soils and water saprophytes do not enter a treatment plant in significant numbers and do not grow naturally in the mixed liquor or sludges. Such conditions may contribute to decreased efficiencies of wastewater treatment, plant upsets, and numerous operational problems (1).

To improve or correct the inability of the treatment plant's biota to degrade organic wastes efficiently, many wastewater treatment facilities utilize a program of bio-augmentation, bacterial augmentation, or biomass enhancement. Bio-augmentation is the addition of specific strains of microorganisms to the wastewater treatment units. These microorganisms can be either selected naturally occurring strains of bacteria or genetically altered or mutated bacteria. These bacteria, once introduced in a wastewater system, it is claimed, can improve the performance of the biological systems, can protect and enhance the activity of naturally occurring strains and permit optimum balance and control (1,2).

Although many successful applications of bio-augmentation have been reported, most of these applications refer to the handling of accidental hazardous spills. There is very little information regarding the effects of bio-augmentation on the performance of wastewater treatment system and on the kinetic parameters of the treatment processes.

Chambers (4) investigated the use of the bio-augmentation technique for treatment of dairy wastewater in three different systems: an extended aeration unit, an aerated lagoon and an oxidation ditch. In the extended aeration unit, prior to bio-augmentation, the removal efficiency in terms of BOD (biochemical oxygen demand) and COD (chemical oxygen demand) ranged from 70 to 85%. After application of bio-augmentation, BOD removal efficiency increased to 98% and COD removal efficiency increased to 91%. The sludge settleability was also improved. In the aerated lagoon system, a 5 day BOD removal efficiency was 81% and effluent discharge was well above the 50 lb. BOD/day permit requirement prior to bio-augmentation. After 6 weeks of bio-augmentation, the BOD removal efficiency increased to 98%. In the oxidation ditch system, bulking suspended solids were encountered. During 9 months of bio-augmentation, BOD and suspended solids removal efficiency increased to 98%. Withdrawal of bio-augmentation resulted in deterioration of effluent quality.

Qasim and Stinehelfer (5) examined the kinetic parameters resulting from a special type of dried bacterial addition, using bench-scale continuous flow reactors. The bacterial culture product tested had some effect on the sludge growth yield coefficient and the decay coefficient, but it had no effect on the substrate utilization coefficient and the substrate concentration at half of the maximum growth rate.

A study was conducted in the Springfield sewage treatment plant, Springfield, Massachusetts, to compare the energy utilization between two aeration basins, one with naturally developed bacterial population without LLMO addition, and the other with LLMO addition (6). The results indicated that with nitrate concentrations as a control parameter, the wastewater treatment could achieve a 20% reduction in energy costs with LLMO bio-augmentation.

Martin and Zell (7) examined the effectiveness of bio-augmentation in the treatment of dairy processing wastewaters. Neither of the commercially available cultures was effective in prevention of the proliferation of filamentous microorganisms and the resulting development of sludge bulking conditions.

The objectives of this study are to determine the effects of the application of bio-augmentation on the activated sludge treatment of weak strength municipal wastewaters, and to determine the bio-kinetic parameters.

MATERIALS AND METHODS

Feed Wastewater

Primary effluents taken from Southerly wastewater treatment plant, Cleveland, Ohio, U.S.A., were used as feed in this study. The wastewater had low organic strength with an average TOC of 19.3 mg/l and an average COD of 68.1 mg/l during the study period.

Complete-Mix Activated Sludge Reactors

A system consisting of six continuous flow bench scale activated sludge reactors was used for the research. Each reactor consisted of two parts: the aeration tank, having a required volume, and the settling tank for the settling of the solids present. An inclined plate facilitated the return of settled sludges to the aeration tank. A movable baffle separated the two tanks. Two of the reactors had an aeration tank volume 0.5 l, two of 1 l, and two of 2 l. Aeration, controlled by a manifold, and injected through air diffusers, provided the oxygen requirements and the proper mixing in the aeration tanks. A pump was used to transport the influent wastewater from the influent tanks to the aeration tanks. The effluent of the settling tanks was collected in the effluent tanks.

The bench scale aerobic reactors were seeded with waste activated sludge taken from a municipal wastewater treatment plant in the Greater Cleveland area, Cleveland, Ohio. The acclimation period was about 2 months prior to the study using a fill and draw batch operated procedure. The acclimation was monitored by oxygen uptake rate measurement. The acclimated seed was then used in the activated sludge reactor.

Bacterial Culture Product Addition

Specialized cultures of bacteria or prepackaged enzyme preparations have been available for addition to wastewater treatment facilities for more than 30 years. From the large number of commercial bacterial culture products, the LLMO (liquid live microorganisms), manufactured by the General Environmental Science Corporation, Cleveland, Ohio, was used in this study. LLMO consists of naturally selected bacterial strains which are commonly found in soil and aquatic environments and are not genetically altered or mutant bacteria. In contrast to other commercially available products which consists of unspecified types of bacteria, the LLMO strains are well known. Evaluation of biological treatment performance resulting from a specified bacterial population will be more meaningful than that resulting from unspecified cultures and the conclusions drawn will be more valid.

The LLMO strains have been selected because of their capabilities to decompose organic matter and to crossfeed upon the by-products of one another's metabolisms. They are preserved in a liquid suspension and consist of seven strains: bacillus, pseudomonas, nitrosomonas, nitrobacter, cellulomonas, enterbacter, and rhodopseudomonas. Bacillus produces exoenzymes to degrade protein, fats and starch to a point where they can be metabolized both by itself and by other bacteria. Pseudomonas is able to degrade a wide variety of organic compounds in addition to performing denitrification, the conversion of NO_3 to N_2. Nitrosomonas converts ammonia to nitrite in nitrification. Nitrobacter converts nitrite to nitrate in nitrification. Cellulomonas degrades cellulose, the major component of plant materials, at a relatively slow rate. Enterobacter grows aerobically on a wide variety of organic compounds and produces a red pigment, when grown anaerobically in the presence of light (photosynthesis). This characteristic provides an easy quality control on the product because the pigment is intracellular. If upon shaking the product the reddish color is evenly distributed through the solution, the cells may have lysed.

Run Protocols of Continuous Reactor Study

The run protocols of the continuous reactor study are shown in Table 1. The hydraulic detention time was 6, 12, and 24 hours, for reactors 1 and 4, 2 and 5, and 3 and 6, respectively. Reactors were operated at a sludge age of 20 days. Reactors 1, 2 and 3 were operated as bio-augmented reactors with addition of LLMO at a dosage of 10 mg/l per day, while reactors 4, 5 and 6 were operated as control reactors without bio-augmentation. Because of the very low organic content in the influent with an average TOC of about 20 mg/l, the mixed liquor suspended solids concentration in the aeration tank of the activated sludge reactor was decreasing and reached as low as 300 mg/l. Therefore, the sludge wastage was stopped after the 29 day of the continuous reactor run. Even without the sludge wastage, the MLVSS concentration did not increase significantly.

Reactors were operated for about 3 times the sludge age to reach pseudo-steady state operation condition before initiation of reactor sampling. Samples were taken from reactor feed and effluents for measurement of TOC, COD, pH, and TSS (total suspended solids), NH_3-N and NO_3-N. All analytical procedures followed those listed in the Standard Methods (8). The TOC analysis was conducted by an O & I TOC analyzer. The NO_3-N test was conducted by Bausch and Lomb Spectrokit Reagent System - Cadmium reduction. The NH_3-N test was also conducted by Bausch and Lomb Spectrokit Reagent System - Nesslerization method.

TABLE 1
Run Protocol

Reactor #	HDT (Hrs)	θ_c (Days)	LLMO (mg/l)
1	6	20	10
2	12	20	10
3	24	20	10
4	6	20	0
5	12	20	0
6	24	20	0

RESULTS AND DISCUSSIONS

The activated sludge reactor performance data is summarized in Table 2. The weak strength municipal wastewaters contained 68 mg/l TSS, 43.3 mg/l VSS (volatile suspended solids), 19.3 mg/l TOC, and 68.1 mg/l COD with a pH of 6.28. Due to the low organic strength of the wastewaters, the long hydraulic detention time in the aeration tank, and the long sludge age used in this study, there was little difference in the treatment performance between the augmented reactors and the control reactors.

TABLE 2
Summarized Results of Reactor Performance

Parameter	Influent	Reactor #					
		1	2	3	4	5	6
Flow Rate (1/d)		1.77	1.69	1.82	1.84	1.76	1.81
HDT (Hrs)		6.92	14.50	26.93	6.63	14.01	26.99
MLSS (mg/l)		911	1639	1036	1115	1092	1265
MLVSS (mg/l)		571	967	582	703	639	708
TSS (mg/l)	68	43.6	24.4	50.2	30.2	71.1	40.4
VSS (mg/l)	43.5	24.3	12.9	27.3	17.6	40.0	24.8
TOC (mg/l)	19.3	10.3	9.9	10.1	9.8	10.4	9.4
COD (mg/l)	68.1	28.4	24.1	23.6	24.1	28.2	23.8
F*/M (D^{-1})							
TOC ($x10^{-3}$)		3.88	0.82	0.92	2.51	1.84	0.68
COD ($x10^{-3}$)		15.54	3.47	3.44	11.19	6.07	3.25
NH_3-N (mg/l)	41.2	9.6	8.0	8.2	9.3	9.3	10.4
NO_3-N (mg/l)	4.0	12.7	15.9	22.8	11.2	19.0	21.0
D.O. (mg/l)		6.79	6.49	7.08	6.35	6.57	7.04
O2 Upt. Rate (d^{-1})		0.64	0.15	0.15	0.36	0.20	0.12
ZSV (m/Hr)		3.61	1.41	2.68	3.03	2.78	2.62
SVI (mg/l)		205	282	260	203	208	250
pH	6.28	6.47	6.40	6.40	6.47	6.42	6.39

Organic Removal

Because of the very low organic strength of the influent, the maximum possible degradation took place in hydraulic time much shorter than those used in this study. The average values of each reactor effluent in terms of TOC was about 10 mg/l and in terms of COD about 25 mg/l. The average TOC and COD removal efficiency was about 50 and 60% respectively. Figures 1 and 2 depict the TOC, and COD removal efficiency versus hydraulic detention time in the reactor, respectively.

FIGURE 1
Average % TOC Removal vs HDT

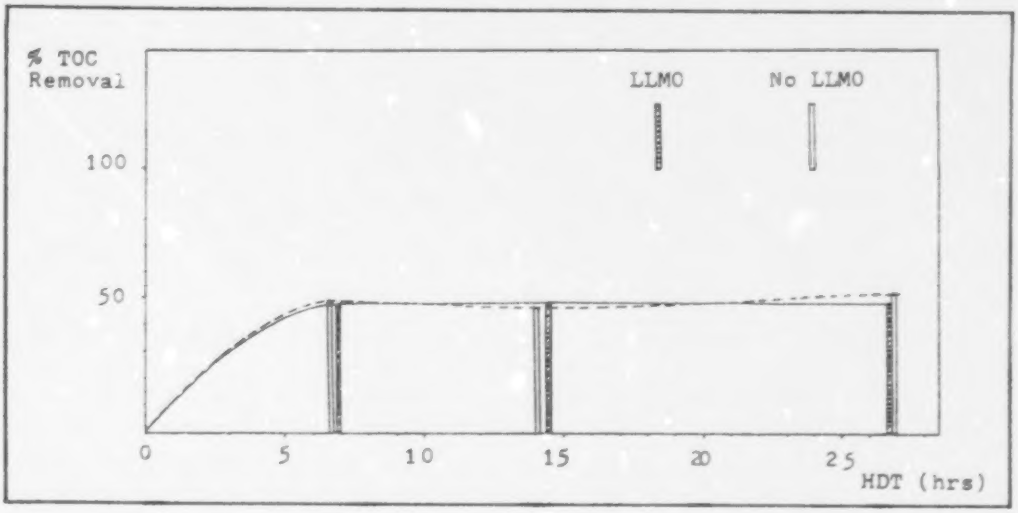
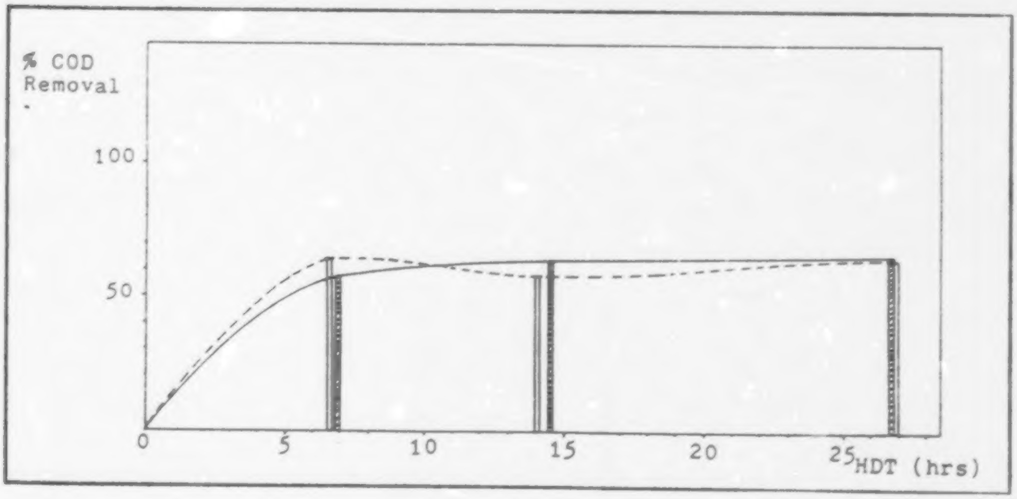


FIGURE 2
Average % COD Removal vs HDT



Although there is little difference in the percent of organic removal according to hydraulic detention time, there is noted difference in the amount of organic removal per unit of MLVSS or food to microorganisms ratio (F/M ratio) according to hydraulic detention time. Figure 3 and 4 shows the F/M ratio in terms of TOC versus hydraulic detention time, and the F/M ratio in terms of COD versus hydraulic detention time, respectively. For both cases, the F/M ratio was decreasing according to hydraulic detention time. This implies that more amount of organics was removed in shorter hydraulic detention time per unit of solids concentration. The bio-augmented reactors 1 and 3 performed better than the corresponding non-bioaugmented reactors 4 and 6, while the bio-augmented reactor 2 did not perform as well as the non-bioaugmented reactor 5. The average F/M values of the LLMO reactors, i.e. the bio-augmented reactors, were 0.00187 per day in terms of TOC and 0.00748 per day in terms of COD, compared to the average values of the non-LLMO reactors, i.e. the non-bioaugmented reactors, which were 0.00168 per day in terms of TOC and 0.00684 per day in terms of COD. In both cases the LLMO reactors exhibited about 10% better performance.

FIGURE 3
Average F*/M (TOC) vs HDT

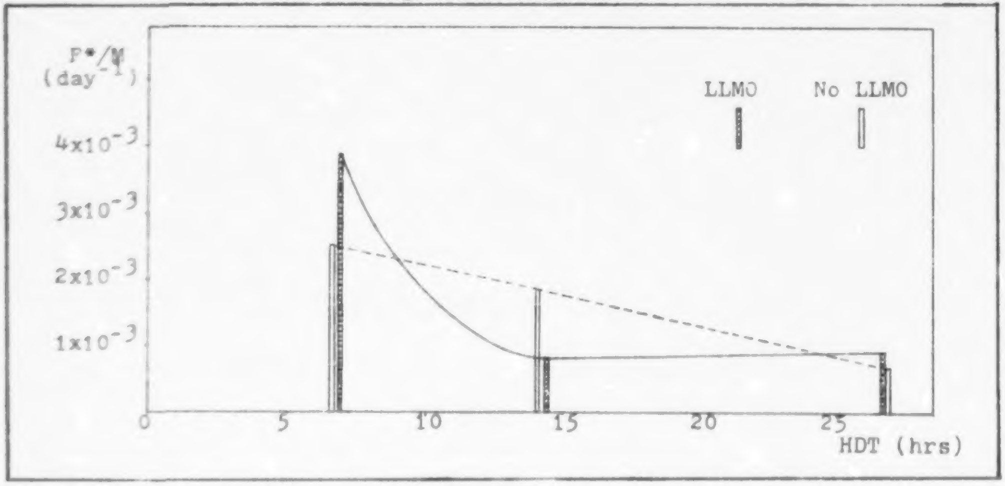
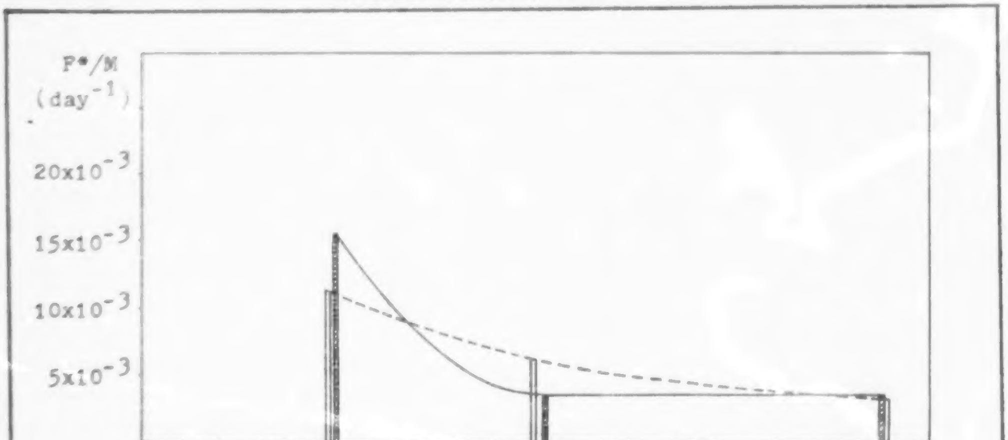


FIGURE 4
Average F*/M (COD) vs HDT



The TOC removal coefficients were 0.00128 for reactor 1; 0.00058 for reactor 2; 0.00090 for reactor 3; 0.00098 for reactor 4; 0.00080 for reactor 5; and 0.00068 for reactor 6. The corresponding COD removal coefficients were 0.00142, 0.00054, 0.00048, 0.00114, 0.00094, and 0.00040. Relationship between TOC and COD removal coefficient and hydraulic detention time is shown in Figures 5 and 6. With the exception of reactor 2, the general trend indicated that the organic removal rate coefficients decreased with increasing hydraulic detention time. Overall, the LLMO reactors had slightly higher organic removal rate coefficients than the non-LLMO reactors.

FIGURE 5

TOC Removal Rate Coefficients vs HDT

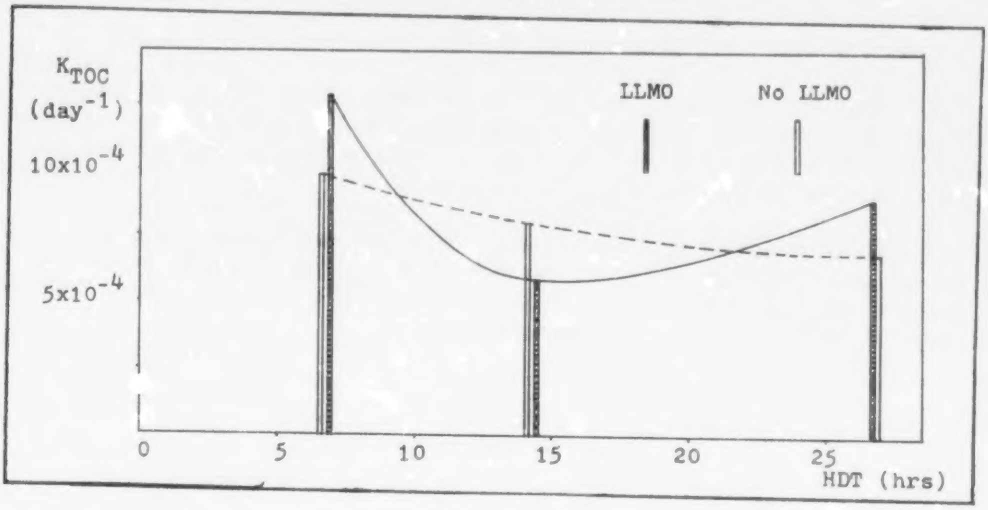
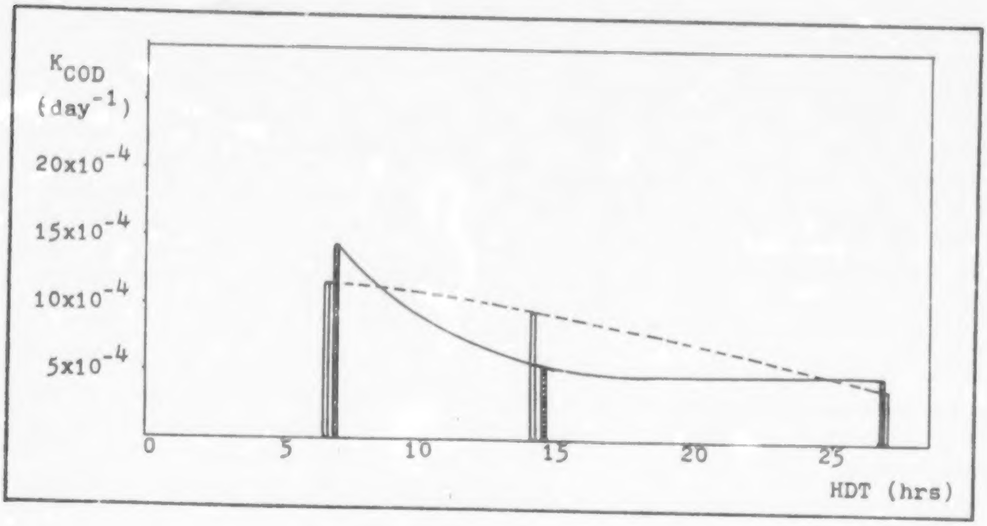


FIGURE 6

COD Removal Rate Coefficients vs HDT



Sludge Production

A mass balance around the reactor yields the following equation:

$$\Delta X_v = X_v\ [\text{out}] - X_v\ [\text{in}] + \Delta\ \text{MLVSS}$$

where: ΔX_v = net amount of sludge produced [mg]

X_v [in] = total amount of VSS in the influent [mg]

X_v [out] = total amount of VSS in the effluent + total amount of VSS wasted [mg]

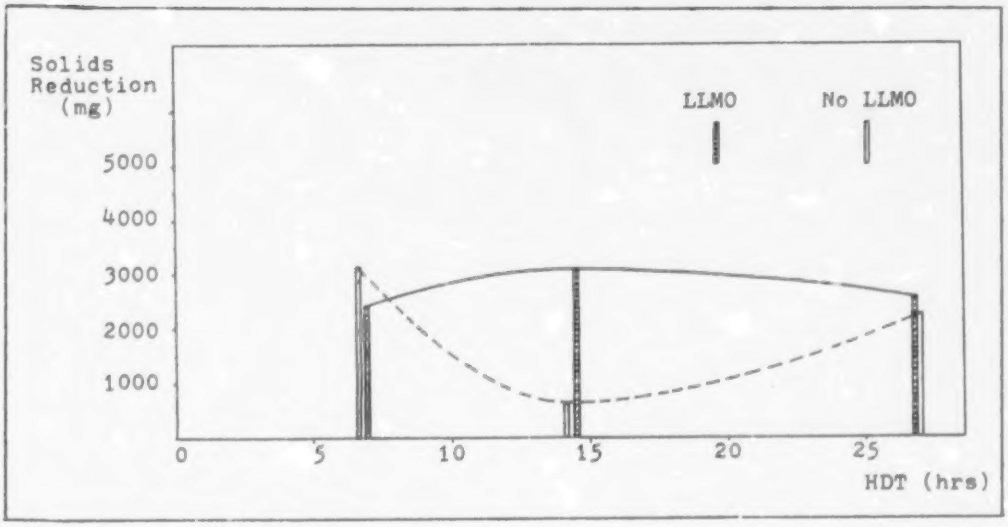
Δ MLVSS = -[(Initial MLVSS) - (Final MLVSS)]

The amount of sludge production was found to be -2382, -3079, -2536, -3124, -591, and -2233 mg/day for reactor 1, 2, 3, 4, 5, and 6 respectively. The sludge production in all reactors was negative. This is due to the very low organic content in the feed wastewater which could not support an increase in bacterial population or growth.

Figure 7 depicts the amount of sludge reduction versus hydraulic detention time in the reactor. The average amount of sludge reduction was 2666 mg/day, and 1983 mg/day, for the LLMO reactors, and non-LLMO reactors, respectively. This implies that bio-augmented reactors produced 34.4% less amount of sludge than the non-bioaugmented reactors.

FIGURE 7

Solids Reduction vs HDT

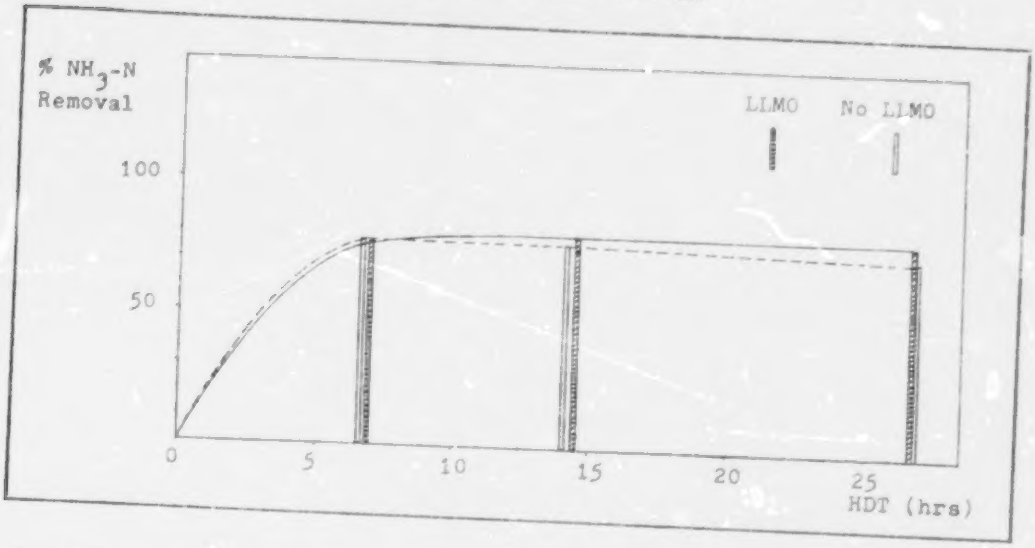


Oxygen Requirements

All the reactors were operated with high dissolved oxygen concentrations varying between 6 and 7 mg/l. The oxygen uptake rate is the amount of oxygen consumed per unit mass of micrcorganisms per unit of time. It is plotted versus hydraulic detention time in Figure 8. The amount of organic matter removed per unit mass of microorganisms per unit of time, i.e. the F/M ratio, and the amount of oxygen consumption are directly related.

FIGURE 8

Average % Ammonia Removal vs HDT



Nitrification

The ammonia concentration was reduced from an influent concentration of 41.2 mg/l to an effluent concentration of 8 to 10 mg/l with a corresponding removal efficiency of about 80%. Figure 9 shows the percent of ammonia removal efficiency versus hydraulic detention time. There is little difference in the ammonia removal efficiency according to hydraulic detention time and LLMO addition. However, there is a difference in nitrate production according to hydraulic detention time. More nitrate was produced at longer hydraulic detention time. This means that more nitrogen was used at shorter hydraulic detention time for cell synthesis, while more ammonia was oxidized to nitrate at longer hydraulic detention time. The nitrate concentration average 17.1 mg/l for the effluents in both LLMO and non-LLMO reactors. Effluent nitrate concentration is plotted versus hydraulic detention time in Figure 10.

pH

The average pH value of influents was 6.28, while the effluent pH values were slightly higher. The average pH value was 6.47, 6.40, 6.40, 6.47, 6.42 and 6.39 for reactor 1, 2, 3, 4, 5, and 6, respectively. The average value for all reactors was more or less the same. The small decrease in pH according to hydraulic detention time is due to the higher amount of nitrate produced from ammonia oxidation.

FIGURE 9

NO_3-N Concentrations vs HDT

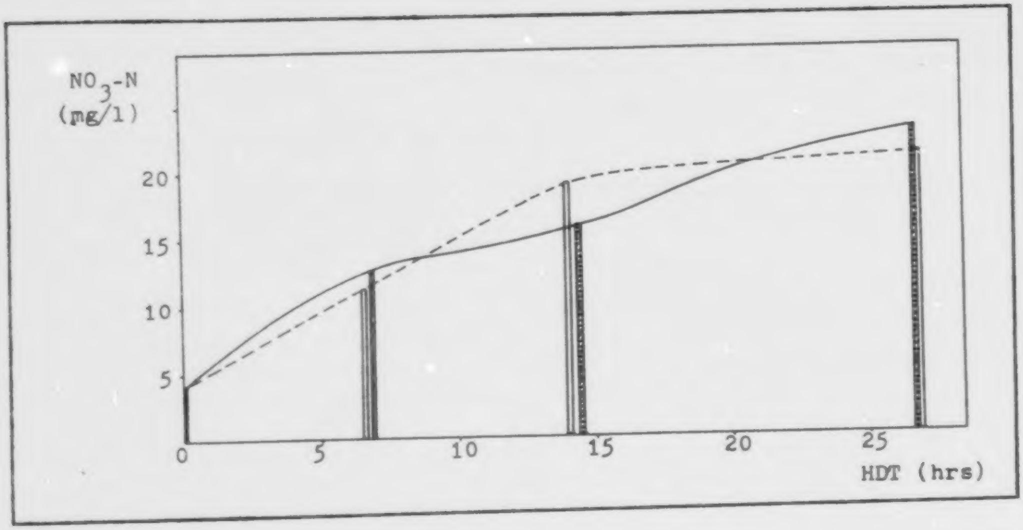
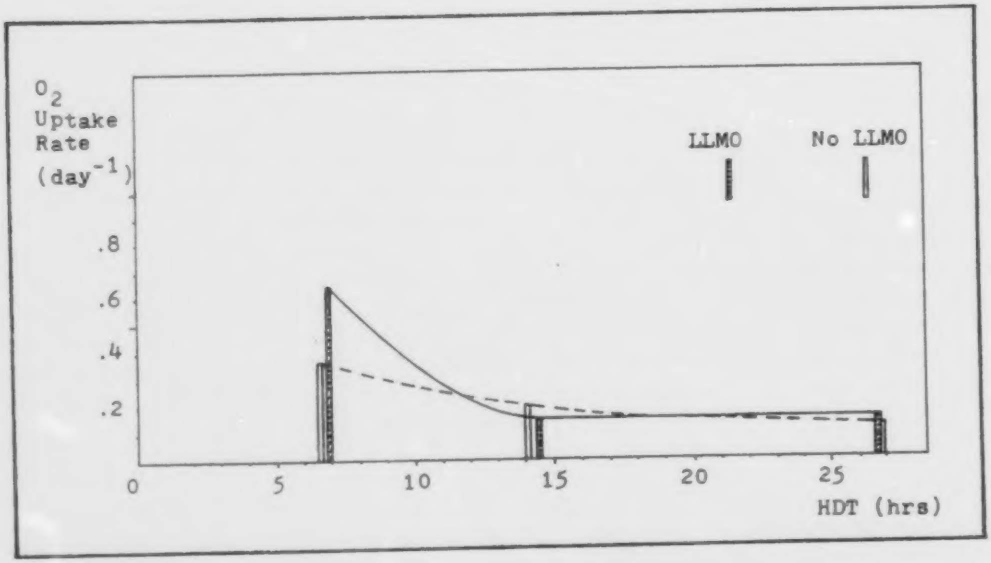


FIGURE 10

O_2 Uptake Rate vs HDT



Sludge Settling

The zone settling velocity (ZSV) is obtained bv the sludge settling tests, and represents the velocity of sludge settling in units of length per unit of time (m/hr or ft/hr). The sludge volume index (SVI) represents the volume of sludge occupied by a unit of mass of solids (ml/g) after a 30 minute settling. The ZSV and SVI is plotted versus hydraulic detention time in Figure 11, and 12, respectively. In general, the non-LLMO reactors produced a sludge with slightly better settling characteristics. This is mainly due to the poor overall performance of reactor 2.

FIGURE 11

Zone Settling Velocity vs HDT

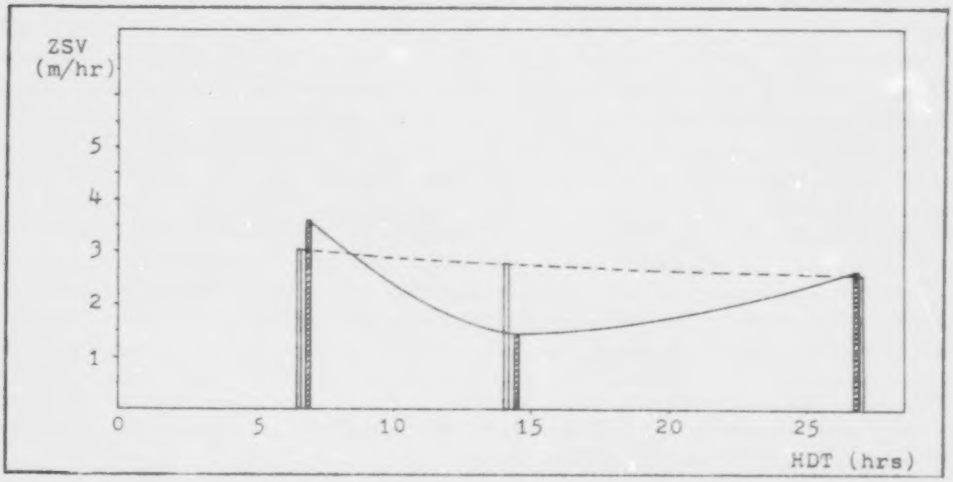
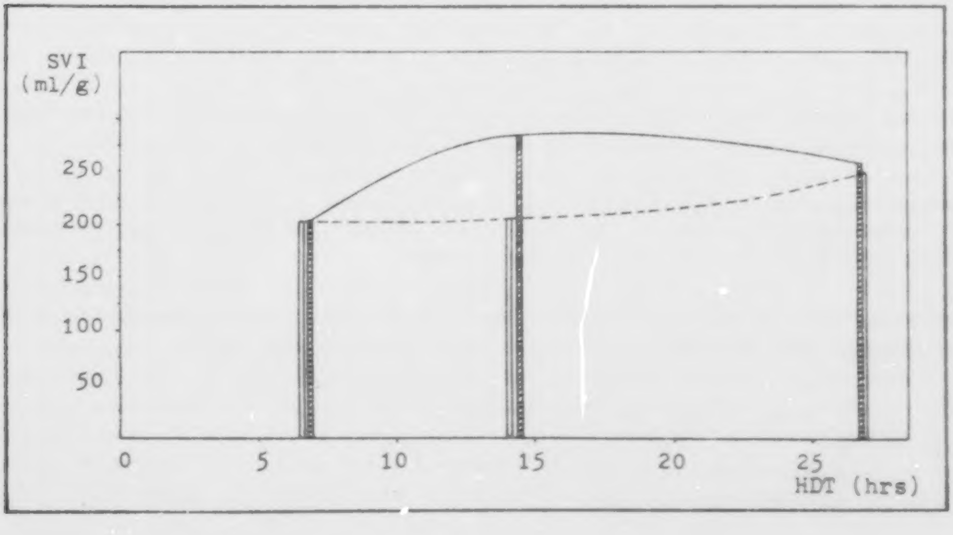


FIGURE 12

Sludge Volume Index vs HDT



CONCLUSIONS

1. For the low strength municipal wastewater, there was little difference in organic removal efficiency between bio-augmented and non-bioaugmented activated sludge reactors.

2. The amount of organic removed per unit of mass of microorganisms (F/M ratio) decreased with increasing hydraulic detention time in the activated sludge reactors. Organic removal coefficients increased with increasing F/M ratio. The oxygen consumption was proportional to organic removal rate.

3. The amount of ammonia removal was the same for all reactors. At shorter hydraulic detention time, more ammonia was used for cell synthesis, while at longer hydraulic detention time, more ammonia was oxidized to nitrate in the nitrification process.

4. Bio-augmentation improved the amount of organic removed per unit mass of microorganisms by 10% and reduced sludge production by 34.4% in the activated sludge treatment of weak organic strength municipal wastewaters.

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BENTONITE AS A COAGULANT AID IN WATER TREATMENT

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INTRODUCTION

References to the use of bentonite in water treatment can easily be traced back in history. A recent article by Lund and Nissen [1986] showed that bentonite could be used to remove viruses from natural water. The interesting point in this study is that the bentonite which was tested has been used by villagers along the Nile River for hundreds of years to clarify their own drinking water supply. Bentonite has also been proven to act as a very effective coagulant in waters which contain high concentrations of divalent cations Ca^{++} and Mg^{++} [Olin and Peterson, 1937; Olin and Gauler, 1938] and the purification of sewage [Olin et al., 1937; Olin et al., 1942]. Application of bentonite to wastewater treatment to achieve positive effects in the removal of phosphorus [Jorgensen et al., 1973], colour and chemical oxygen demand [Rebhun et al., 1969] are also known. Bentonite has been used in an unaltered state to remove soluble aromatic hydrocarbons from drinking water [El-dib et al., 1978] but has also been treated to selectively adsorb organic pollutants from aqueous solutions [Wolfe et al., 1986].

The Buffalo Pound Water Treatment Plant treats the water from Buffalo Pound Lake, 1/2 mile northwest of the plant. The lake is located on the Qu'Appelle River with a length of approximately 20 miles, a width of 1/2 to 3/4 mile and a mean depth of 12 feet. The lake is characterized by high algal growths during summer months and relatively low turbidity year round. The treated water is supplied to the cities of Regina and Moose Jaw in Saskatchewan. The plant itself is a typical surface water treatment facility. Water is pumped to the plant after water pre-chlorination at the lake intake. Upon entering the plant, the influent is dosed with aluminum sulfate, chlorine and, after a short mixing period, a liquid polymer is added prior to the flash-mix tank. After flash mix, the coagulated water flows to one of four clarifiers for flocculant settling before entering the rapid sand filters. Recent additions to the plant include the eight granular activated carbon filters which came on stream in the summer of 1985 to reduce the taste and odor problems encountered during hot summer months. High quality water is presently being produced by the plant but the prospect of cost reduction is always a possibility.

In the past, interest has been expressed with regards to the use of bentonite as a coagulant aid at the Buffalo Pound Water Treatment Plant. Through the use of bentonite in the treatment process at the plant, it was felt that the dosages of aluminum sulfate could be reduced while maintaining a high quality finished water. If this is indeed true, it could result in an overall reduction in cost of treatment due to the price differential existing between aluminum sulfate and bentonite. Some preliminary testing has been done at the plant and by a local consultant to determine if bentonite would in fact be beneficial. The results of this testing did not form a direction of action since both positive and negative results were obtained.

Therefore a preliminary laboratory study was conducted to determine whether the addition of bentonite was beneficial.

OBJECTIVES

The objectives of the study were to investigate the effect of bentonite dosage on the following three aspects:

1. reduction in the amount of aluminum sulfate required to produce favorable conditions for coagulation;

2. enhancement of settling characteristics of aluminum hydroxide flocs; and

3. improvement in the finished water quality.

The scope of the study included laboratory testing for a three day period in February 1986 of pre-chlorinated raw water samples collected from the influent to the Buffalo Pound Water Treatment Plant, using sodium bentonite.

MATERIALS AND METHODS

Bentonite Samples

The sodium bentonite used for testing was manufactured by Avonlea Minerals. The base material is montmorillonite from the Bear Paw formation located south east of Moose Jaw. The sample has been processed to allow 75% passage on a 325 U.S. sieve and has a specific gravity of 2.33. A chemical analysis of the bentonite used is shown in Table 1.

TABLE 1

Chemical Analysis of Avonlea Bentonite

Constituent	Percentage
SiO_2	61.4
Al_2O_3	16.1
Fe_2O_3	3.2
TiO_2	0.1
MgO	2.3
CaO	0.6
K_2O	0.4
Na_2O	2.2
H_2O (−105°C)	9.0
H_2O (+105°C)	5.0

Water Samples

Samples were gathered on February 13, 14 and 15, 1986 from the Buffalo Pound Water Treatment Plant. The samples were taken at a point located upstream of the flash-mix tank. This meant that the samples were pre-chlorinated raw water. The quality of the water remained relatively constant throughout the sampling period and all testing was conducted within 24 hours of the sample time.

Jar Test

The jar test is the standard test used in the evaluation of the relative effectiveness of varying coagulant dosages. One of the greatest drawbacks of the jar test is the temperature variations between plant and laboratory conditions [Neuman, 1981]. This temperature differential will alter the coagulant dosage required to produce good coagulation [Mohtadi and Rao, 1972]. Consideration was given to the idea of constructing water baths to maintain plant-like temperatures but was beyond the time limits of this project. However, all samples were allowed to stabilize at room temperature (18-19°C) prior to testing to eliminate differing conditions between tests.

A standard jar test apparatus was used for the testing. Similar mixing times as are incorporated at Buffalo Pound were used as follows:

1. Rapid mix for 3 minutes @ 100 r.p.m.

2. Slow mix for 27 minutes @ 30 r.p.m.

3. Sedimentation for 30 minutes

A stock solution of aluminum sulfate was prepared by dissolving 10.00 g of $Al_2(SO_4)_3 \cdot 18H_2O$ in 1 L of distilled water. This was then used to create the required dosages in 1L samples used in testing. Six different dosages of 20, 40, 80, 120, 180 and 300 mg/L and one control were used.

It was found that the addition of the bentonite in powdered form resulted in mixing problems. For ease of application, the bentonite was maintained in a 10 g/L slurry during testing. This slurry was then used to create dosages of 50 and 100 mg/L in the raw water. Previous investigations using bentonite in the coagulation process adopted dosages ranging from 500 mg/L in the treatment of wastewater [Rebhun *et al.*, 1969] to as little as 5 mg/L in water treatment [Allied Colloids Ltd., 1975] with usage dosages between 15 and 100 mg/L [Twort, 1985].

Each sample was tested in its natural state and using two different dosages of bentonite. The bentonite was suspended in the sample prior to addition of alum to allow for total involvement in the coagulation process.

Zeta Potential (ZP)

The colloids that we are attempting to remove have a definite electrostatic potential which in fact keeps them from coagulating. If the particles are placed in an electric field, the positive particles will migrate to the negative pole while the negative particles will migrate to the positive. The speed at which the particles travel in the induced electric field is the electrophoretic mobility and can be used to calculate the ZP of the particle [Letterman *et al.*, 1982].

Although a rather simple concept, actual measurement of ZP was very time-consuming when Black first used ZP to study the coagulation of water impurities in 1948 [Swope, 1977]. It was not until the early 1960's that the Zeta-Meter was developed by Dr. Thomas Riddick, making measurement both easy and rapid [Riddick, 1961].

The Zeta-Meter was the device used to determine the ZP of the different solutions in this study. After approximately one minute of rapid mix in the jar test, samples of 50 ml were drawn from a depth of 2 cm below the water surface. These seven samples were then used to determine the ZP of each combination of alum and bentonite. The determine of ZP for all seven dosages being analyzed was conducted within 40 minutes of sample draw-off.

Other Tests

pH was measured using a Fischer Accumet pH Meter model 620. Total alkalinity was determined by titrating the sample with standard HC1 to a pH value of 4.5 [American Public Health Association, 1981]. Turbidity was determined before and after treatment to determine the relative effectiveness of the bentonite addition. A Turner Nephlophotometer was used to determine turbidity.

RESULTS AND DISCUSSION

Testing was conducted on February 13, 14 and 15, 1986. Quality of samples remained very constant during the test period which resulted in very similar results for all three days. Tables 2, 3 and 4 show the results.

TABLE 2 - Test Results (February 13, 1986)

BENTONITE (mg/l)	ALUM DOSE (mg/l)	TIME (S)	ZP (mv)	HC1 VOLUME (ml)	TURBIDITY (NTU)	pH	ALKALINITY (AS $CaCO_3$)
0	0	8.30	-14.80	10.7	2.6	7.9	214
0	20	8.90	-12.80	9.9	1.1	7.8	198
0	40	9.40	-11.80	9.4	0.6	7.5	188
0	60	10.30	-11.00	8.8	0.6	7.4	176
0	120	16.10	-7.30	7.4	0.7	7.1	148
0	180	20.00	-5.80	6.1	1.2	6.9	122
0	300	40.10	7.40	3.7	1.0	6.5	74
50	0	8.20	-13.90	10.3	12.1	7.9	210
50	20	8.10	-13.90	9.9	0.5	7.8	198
50	40	9.90	-11.30	9.3	0.5	7.4	186
50	60	11.50	-9.80	8.8	0.5	7.3	178
50	120	11.70	-8.60	7.2	0.3	6.9	144
50	180	25.20	-4.80	5.8	0.4	6.7	113
50	300	17.70	6.60	3.6	0.8	6.3	72
100	0	7.80	-14.20	10.3	38.0	7.9	206
100	20	8.60	-13.10	9.6	0.8	7.7	192
100	40	9.20	-12.20	9.1	0.4	7.4	182
100	60	9.90	-11.30	8.5	0.4	7.3	170
100	120	13.20	-9.10	7.2	0.4	7.0	144
100	180	19.60	-5.90	5.9	0.4	6.7	118
100	300	28.70	4.20	3.7	0.7	6.3	74

Voltage = 200 VDC Specific Conductance = 650 micromhos/cm

TABLE 3 - Test Results (February 14, 1988)

BENTONITE (mg/l)	ALUM DOSE (mg/l)	TIME (S)	ZP (mv)	HC1 VOLUME (ml)	TURBIDITY (NTU)	pH	ALKALINITY (AS $CaCO_3$)
0	0	8.20	-13.90	10.20	2.50	8.10	204
0	20	10.20	-11.10	9.80	1.00	7.70	196
0	40	10.30	-11.00	9.30	0.60	7.40	186
0	60	11.50	-9.80	8.80	0.40	7.20	176
0	120	13.50	-8.30	7.30	0.60	7.00	146
0	180	19.80	-5.90	6.00	0.80	6.70	120
0	300	44.00	2.70	3.50	1.10	6.30	70
50	0	8.30	-17.80	10.10	22.90	8.10	202
50	20	9.00	-12.60	8.70	0.60	7.70	194
50	40	9.50	-11.90	9.10	0.40	7.40	182
50	60	10.60	-10.70	8.60	0.40	7.20	172
50	120	13.40	-8.50	7.30	0.50	6.90	146
50	180	26.90	-4.30	6.00	0.50	6.70	120
50	300	26.30	4.40	3.60	0.60	6.30	72
100	0	8.20	-13.90	10.10	29.90	8.30	202
100	20	9.10	-12.60	9.50	0.50	7.70	190
100	40	10.10	-11.20	9.10	0.50	7.40	182
100	60	10.30	-10.40	8.60	0.40	7.20	172
100	120	12.30	-9.40	7.30	0.20	7.00	146
100	190	17.30	-6.30	6.00	0.20	6.80	120
100	300	31.90	3.60	3.60	0.50	6.30	72

Voltage = 200 VDC Specific Conductance = 630 micromhos/cm

TABLE 4 - Test Results (February 15, 1988)

BENTONITE (mg/l)	ALUM DOSE (mg/l)	TIME (S)	ZP (mv)	HC1 VOLUME (ml)	TURBIDITY (NTU)	pH	ALKALINITY (AS $CaCO_3$)
0	0	8.4	-13.7	10.2	2.3	7.9	204
0	20	8.7	-12.8	9.8	0.8	7.7	196
0	40	9.2	-12.4	9.3	0.6	7.4	186
0	60	11.6	-9.7	8.7	0.4	7.2	174
0	120	14.7	-7.9	7.4	0.4	6.9	148
0	180	33.1	-3.5	6	0.5	6.6	120
0	300	28.2	4.1	3.5	0.7	6.3	70
50	0	9.7	-12.8	10.2	23.6	8	204
50	20	9.3	-12.3	9.5	0.5	8.8	190
50	40	9.5	-11.8	9.1	0.4	7.4	182
50	60	11	-10.3	8.6	0.3	7.2	172
50	120	13.4	-8.5	7.3	0.2	6.8	148
50	180	19.7	-5.9	6	0.2	6.7	120
50	300	32.7	3.5	3.6	0.4	6.3	72
100	0	8.6	-13.1	10	48.7	9	200
100	20	9.1	-12.6	9.6	0.5	7.7	162
100	40	10	-11.2	9.1	0.5	7.5	182
100	60	10.6	-10.7	8.6	0.3	7.2	172
100	120	12.4	-9.2	7.2	0.2	6.9	144
100	180	31	-3.9	5.9	0.3	6.7	116
100	300	26.6	4.4	3.5	0.5	6.3	70

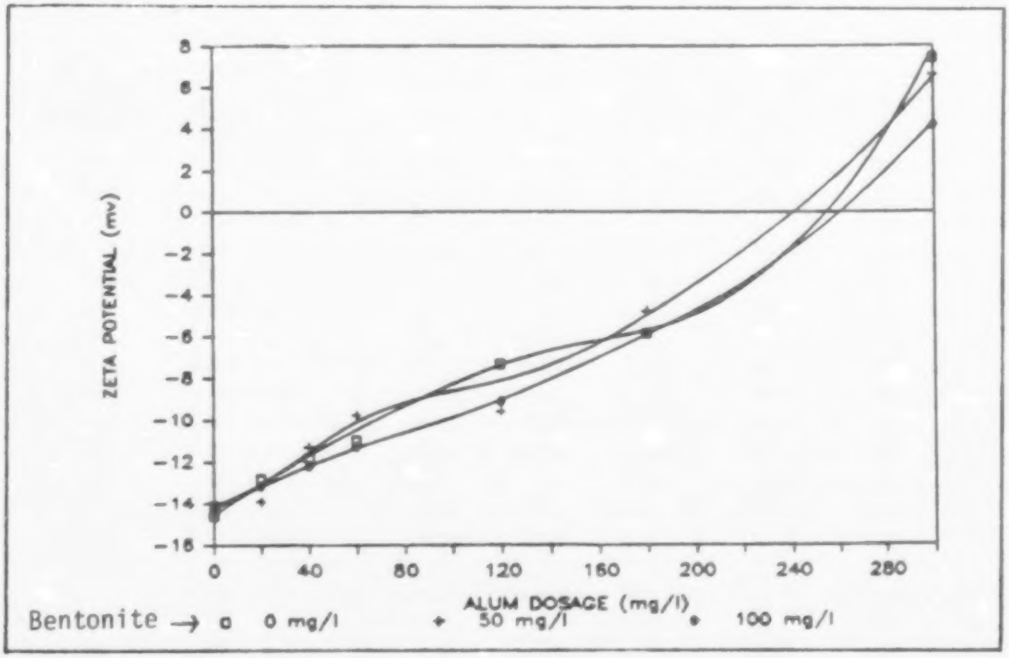
Voltage = 200 VDC Specific Conductance = 680 micromhos/cm

The flocs which formed in the presence of bentonite and alum were noticeably different from those formed with alum alone. The bentonite floc was visibly more compact by not exhibiting the typical light and voluminous texture of the alum floc alone. This is most likely due to the fact that the bentonite has adsorbed some of the hydroxide ions into its structure to create a much tighter bonding and thus a denser floc. The second difference that was noticed was the increased settling velocity when the bentonite was added. For an estimated 80% of the floc to settle, it took approximately 15 minutes in the case of alum alone but only 5 minutes with the bentonite dosage. This aspect of bentonite addition has been mentioned in previous papers [Rebhun *et al.*, 1969; Jorgensen *et al.*, 1973]. In order to obtain a better understanding of the increased settling velocity, column tests should be conducted comparing the settling time under the two conditions.

Figure 1 shows the effect of alum dosage on the zeta potential using the February 13 data. Each of the three curves represents a different bentonite dosage as indicated in the legend. From this graph we can see that the zeta potential was not significantly altered by the addition of the bentonite either in the 50 or 100 mg/L dosages. This is a very important fact since in order to promote coagulation, the zeta potential must be lowered to create instability of the colloids. Therefore, bentonite was not acting as a coagulant aid.

FIGURE 1

Zeta Potential Reduction
Using Various Bentonite Dosages



The addition of bentonite to the samples had no effect on the original alkalinity or on the removal of the alkalinity (see Figure 2). Plant operations could continue as usual with no changes to the chemical treatment to deal with alkalinity.

FIGURE 2

Alkalinity Removal Using Various Bentonite Dosages

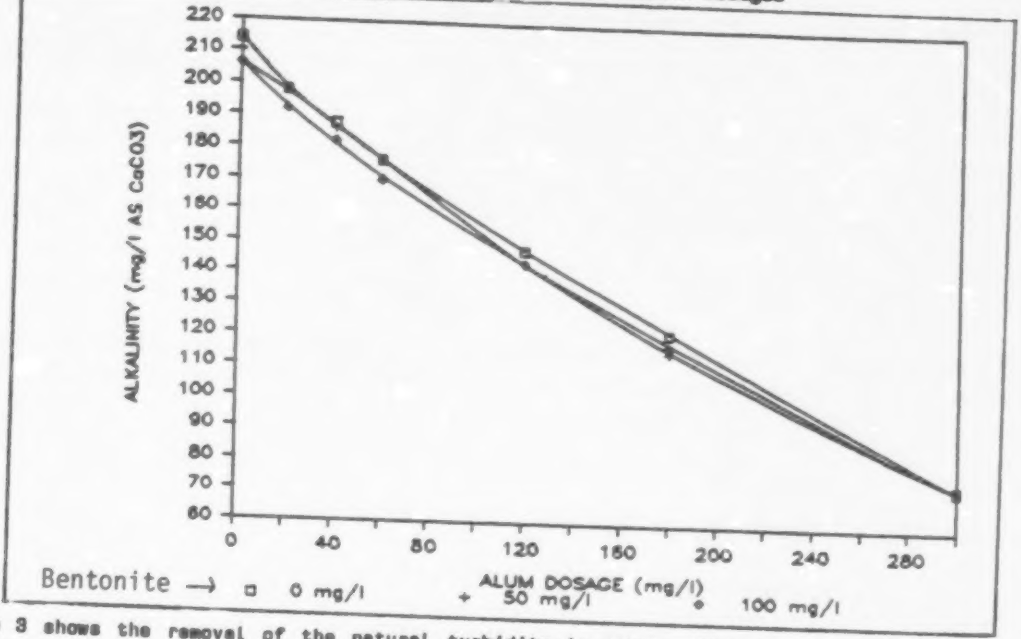


Figure 3 shows the removal of the natural turbidity in water. The removal of turbidity was definitely enhanced through the addition of bentonite.

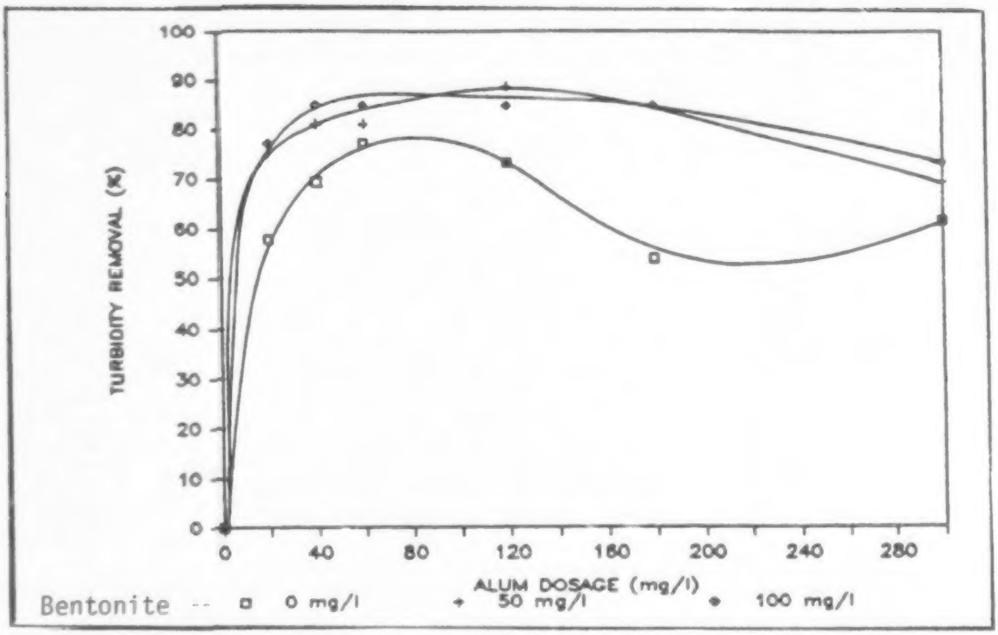
From Figure 3, the following observations were made.

1. The optimum dosage of alum without bentonite is in the range of 60 mg/L which corresponds to the normal dosages used at the treatment plant. [The optimum alum dosage according to the zeta potential (Figure 1) would be in the range of 180 mg/L using Riddick's guidelines].
2. The effect of 50 and 100 mg/L bentonite was basically the same.
3. Turbidity removal is always greater when bentonite is added to the sample. A 10% increase in turbidity removal was experienced at the optimum alum dosage (60 mg/L) when the bentonite was added.
4. As indicated by the relatively flat curves produced when bentonite is added, there is less likely to be an increase in turbidity due to an overdose of alum.

The most interesting point above is point 3. As mentioned, there can be a 10% greater removal through the addition of 50 mg/L bentonite. However, this effect can be magnified if we look at an alum dosage of say 20 mg/L. Referring to Figure 3, a 20 mg/L alum dosage would result in approximately 58% turbidity removal. If 50 mg/L bentonite was first added, the turbidity removal would increase to 77%, the same amount that 60 mg/L alum removed. The net effect is a 66% reduction in alum consumption while maintaining the same effluent quality. Problems of colour removal with alum dosages of 20 mg/L have been experienced. It is probable that with the use of bentonite this problem will be minimized [Rebhun et al., 1989].

FIGURE 3

Removal of Natural Turbidity



Point 1 indicates that although the turbidity removal has been optimised we are not at optimum coagulation as stated by Riddick. It must be remembered however, that the guidelines set down by Riddick are just guidelines. It has only been shown here that for Buffalo Pound Water, optimum coagulation occurs before Riddick's 0 + 5 mv. In our case optimum coagulation occurs at a zeta potential of 10 to 12 mv [Figure 1 @ 60 mg/L alum]. This finding was not unexpected since similar results have been found [Mohtadi and Rao, 1973]. If coagulation dosages were calculated only by the Zeta-Meter results and Riddick's guidelines, inferior water quality would result.

From Figure 1, it was concluded that bentonite was not acting as a coagulant aid. However, the bentonite is providing a benefit as shown by the increased reduction of turbidity. In this case, it appears that the bentonite is acting more as a flocculant aid and weighing agent.

This study was conducted on samples containing relatively low turbidities. During the summer, turbidities increase to an average of 10 NTU in a one month period and maximum values can reach 20 NTU. With the use of bentonite, this greater turbidity could be reduced more easily and at a greater speed to produce less loading on downstream filters. The adsorptive properties of bentonite will also aid in the removal of algae [Avnimelech et al, 1982] during problem time experienced in summer months and thus reduce the organic loading on the granular activated carbon filters.

ECONOMICS

The results of this study indicates an alum dosage of approximately 60 mg/L should be used for optimum turbidity removal. Since the yearly average alum dosage at the plant is 62 mg/L the results of the study can be assumed to represent average plant operation and used to determine the economic impact of the addition of bentonite to the system.

Presently, alum is shipped to the treatment plant in liquid form containing 48.5% aluminum sulfate at a cost of $90 per tonne. The price of bentonite received from Avonlea Minerals was $90 per tonne F.O.B. Wilcox. With a daily average flow of 90.8×10^6 liters/day (20 mgd) the treatment plant produces approximately 3.31×10^{10} liters/year. Using this flow, the difference in cost of treatment can be calculated.

Alum Treatment

At 60 mg/L, a total of 1990 tonnes/year would be required. At 48.5% alum content in the liquid alum, 4100 tonnes of liquid alum per year is required. This amounts to a total cost of $369,000 per year.

Alum and Bentonite Treatment

At 20 mg/L alum, a total of 660 tonnes/year would be required. At 48.5% alum content in the liquid alum, 1360 tonnes of liquid alum per year is required at a cost of $122,400. At 50 mg/L bentonite, a total of 1660 tonnes/year would be required at a cost of $149,400.

The total cost of the alum and bentonite treatment is $271,800. The above calculations show that the treatment with bentonite in conjunction with alum is more cost effective than treatment with alum alone. A total savings of $97,200 or 26% of the average yearly chemical costs could be attained.

It is to be borne in mind that this economic analysis is based on limited testing, assuming that the alum and bentonite in the range used would effectively deal with colour removal. A more detailed testing program is necessary to provide a realistic estimate of dosages required. This economic analysis has also not taken into account the capital and operating costs of a bentonite storage and feed system. However this study has shown the potential aid of the application of bentonite as an aid to flocculation.

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FIVE YEARS OF EXPERIENCE WITH BIOLOGICAL PHOSPHORUS AND NITROGEN REMOVAL

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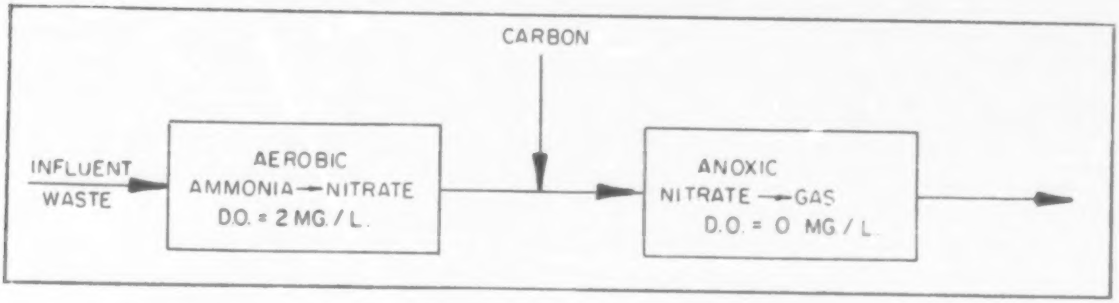
City of Kelowna

Nitrogen Removal

The removal of nitrogen from domestic wastewaters requires a two step biological reaction [Figure 1]. The ammonia-nitrogen from the incoming waste is oxidized in the presence of dissolved oxygen and sufficient organisms to the nitrate-nitrogen form. The oxidation step however uses all available rapidly biodegradable carbon and leaves little carbon for the de-nitrification step. The second step requires a carbon source to reduce nitrate-nitrogen to a gas in the absence of dissolved oxygen and release it to the atmosphere. This reaction can proceed at two very different rates depending primarily on the availability of rapidly biodegradable organics. To increase the speed of the de-nitrification step an expensive chemical addition has often been used to provide a carbon source.

FIGURE 1

Nitrogen Removal External Carbon



In the BARDENPHO process [Figure 2] the two step nitrogen removal process remains however the two steps are reversed in their order by the use of internal recycle pumps. This allows the incoming carbon to react with the nitrates and rapidly complete the second step. This two step process could easily remove 80% of the incoming nitrogen however in the Kelowna plant a second de-nitrification system was added to further increase nitrogen removal and provide sufficient capacity for the unknowns of winter operation at temperatures as low as 8°C. At the time of design it was thought a short aerobic zone was necessary prior to the secondary clarifiers thus the re-aeration zone was added and the 4 stage nitrogen removal was constructed [Figure 3].

The reversal of the first two steps in the process results in a stable operation with the de-nitrification step also removing considerable organic carbon and reducing the load on the aeration system in the aerobic zone. At the Kelowna plant the removal of 90% or greater of the incoming nitrogen has caused very few operational problems. Once the correct sludge age was known and the wasting system adjusted, the only times total effluent nitrogen has exceeded 6 ppm was due to process experimentation or equipment failure [Figure 4].

FIGURE 2
Nitrogen Removal Influent Carbon

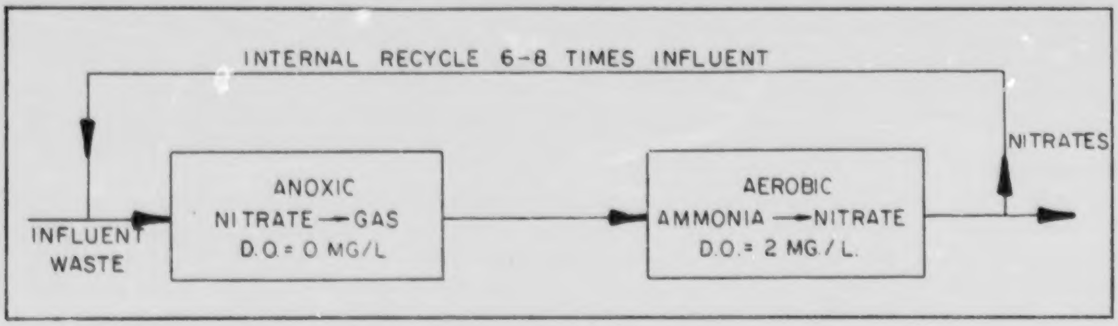

INTERNAL RECYCLE 6-8 TIMES INFLUENT
NITRATES
ANOXIC
NITRATE → GAS
D.O. = 0 MG/L
AEROBIC
AMMONIA → NITRATE
D.O. = 2 MG./L.
INFLUENT WASTE

FIGURE 3
4-Stage Nitrogen Removal

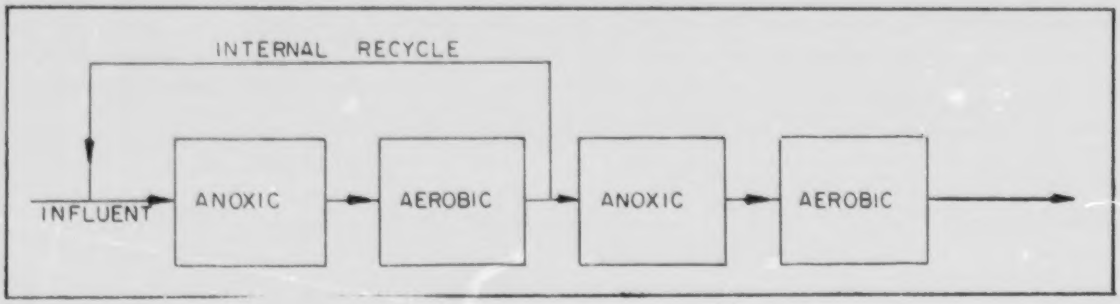
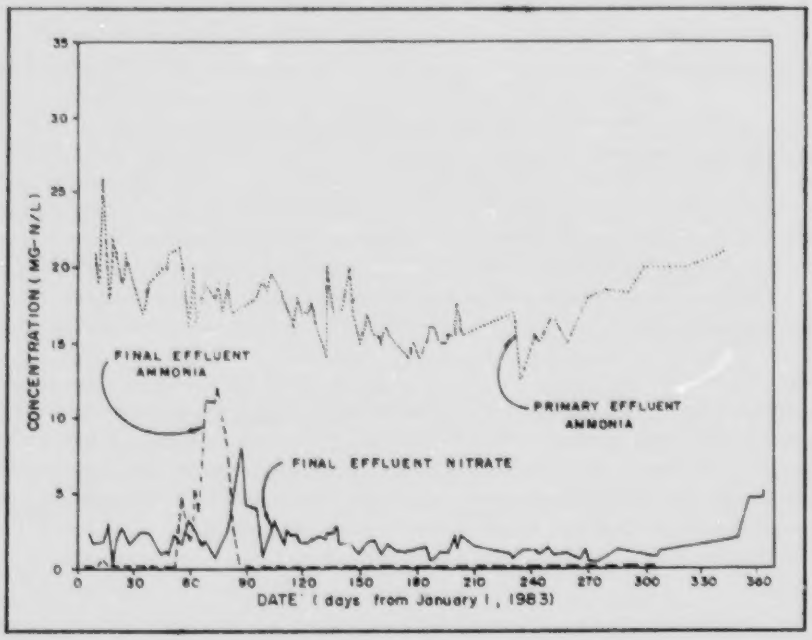

INTERNAL RECYCLE
INFLUENT
ANOXIC
AEROBIC
ANOXIC
AEROBIC

FIGURE 4
Primary and Final Nitrogen



Phosphorus Removal

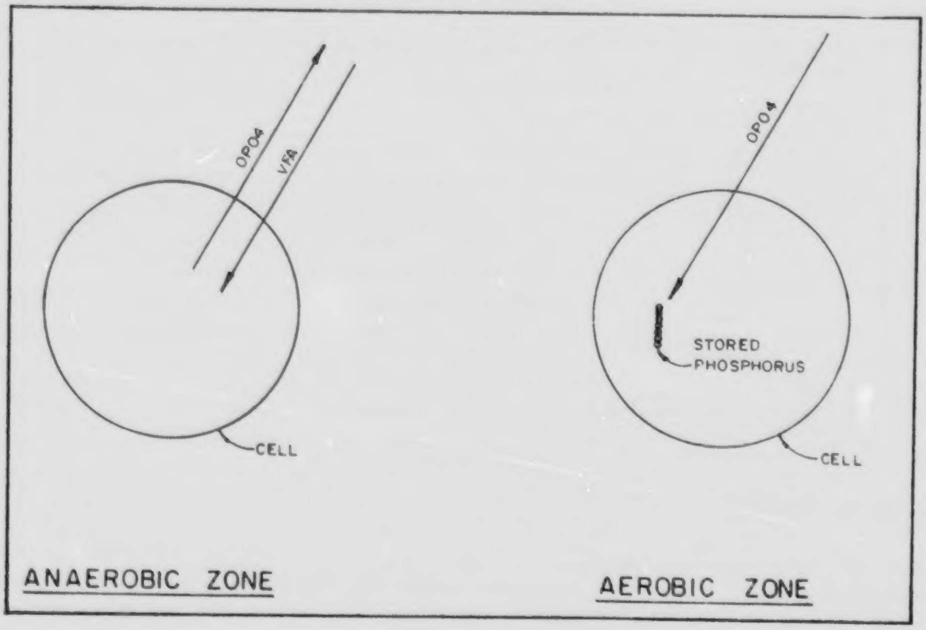
The removal of phosphorus from domestic wastewaters traditionally has resulted in the use of chemicals for the precipitation of the phosphorus and removal in the waste sludge. The BARDENPHO process also results in the removal of the phosphorus in the waste sludge however the storage mechanism appears to be biological and not chemical. The addition to the 4 stage nitrogen removal system (Figure 3) of an anaerobic fermentation zone results in the conditioning of the activated sludge to biologically store an excess of normal metabolic needs. If the conditions in the anaerobic zone are correct the Kelowna facility has demonstrated the activated sludge system has the ability to achieve very high removals of phosphorus.

1. Anaerobic Zone

The anaerobic zone was first used by Dr. J.L. Barnard when he noticed a substantial increase in the activated sludge's ability to remove phosphorus. In the anaerobic zone the dissolved phosphorus concentrations increased higher than could ever be explained by the incoming waste and return sludge. Theories have been developed which satisfactorily explain the exchange of phosphorus for rapidly biodegradable organics and specifically volatile fatty acids (Figure 5).

FIGURE 5

Phosphorus Exchange



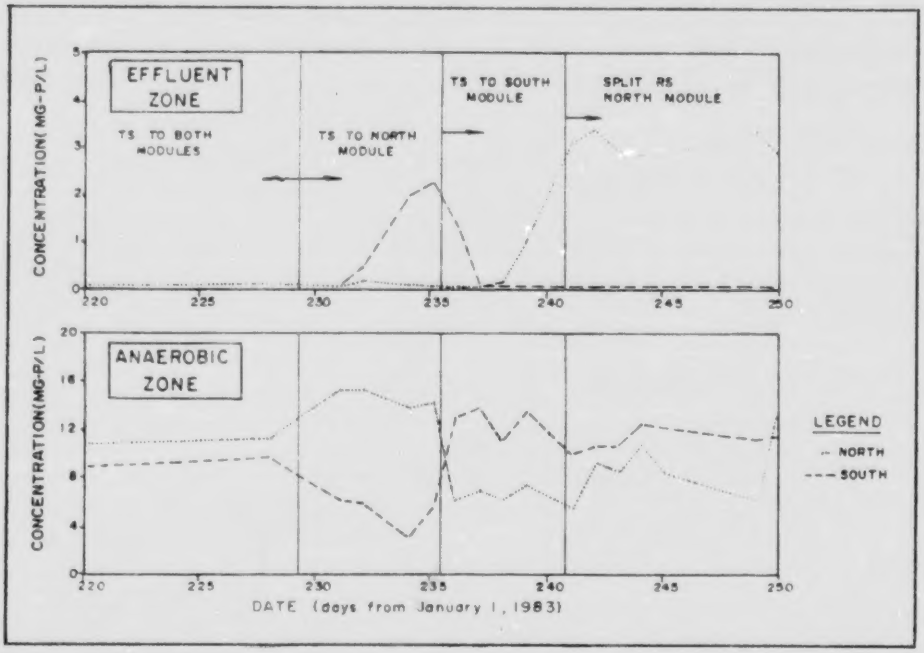
2. Volatile Fatty Acids

In the Kelowna plant the source of the volatile fatty acids has been the gravity sludge thickener. To demonstrate this an experiment was developed where the thickener overflow was diverted first to one side of the process then the other and finally returned equally to both

(Figure 6). The graphs show the increase in ortho-phosphorus concentrations on the side receiving all the thickener overflow and also the decrease in the final effluent levels. When the overflow is diverted to the other side the same effect occurred and when the overflow is given back to both sides the process stabilizes. This convinced us the stimulant for biological phosphorus removal definitely was concentrated in the thickener overflow. Analysis for volatile fatty acids revealed an average concentration of 160 - 200 ppm as acetate. Subsequent experimentation with pure acetate at the University of B.C. and the Kelowna plant confirmed that the acetate fraction of the volatile fatty acids was the most important ingredient necessary to stimulate the phosphorus uptake.

FIGURE 6

Thickener Supernatant Test



3. Split Return Sludge

To increase the availability of volatile fatty acids for the activated sludge in the anaerobic zone the presence of dissolved oxygen and nitrate nitrogen must be reduced to a minimum. One of the sources of both is the return sludge from the secondary clarifiers. It was postulated the return sludge could be split and the result would be half the nitrates and dissolved oxygen and also the activated sludge would be exposed to twice the normal concentration (Figure 7). An experiment was to run to test the theory however it was found that split return sludge only marginally increased the phosphorus removal (Figure 8).

FIGURE 7

Return Sludge Split

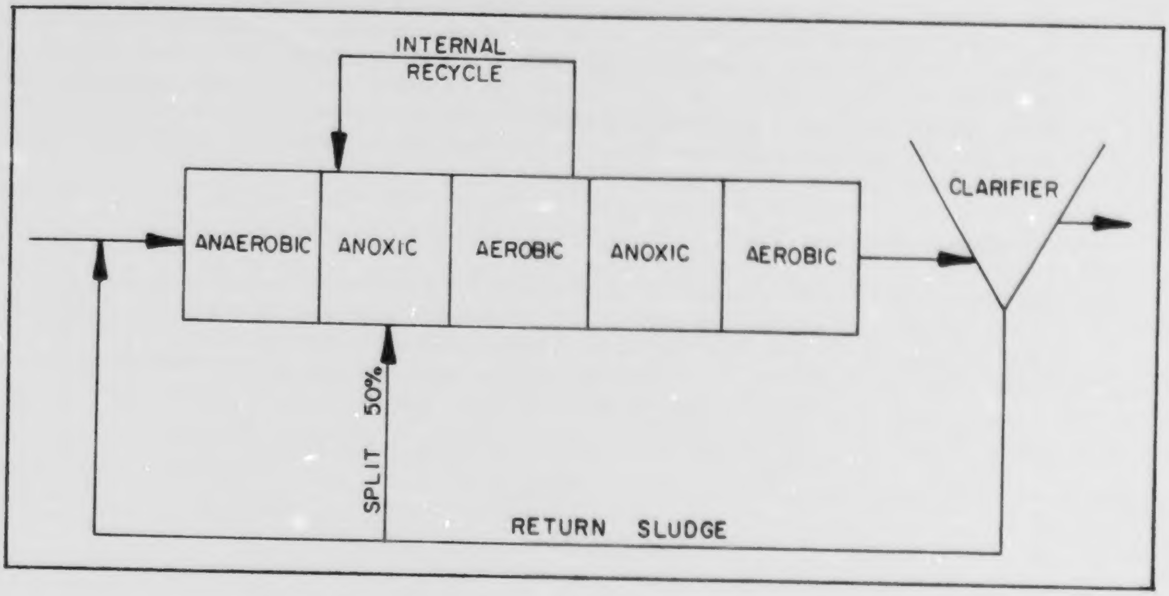
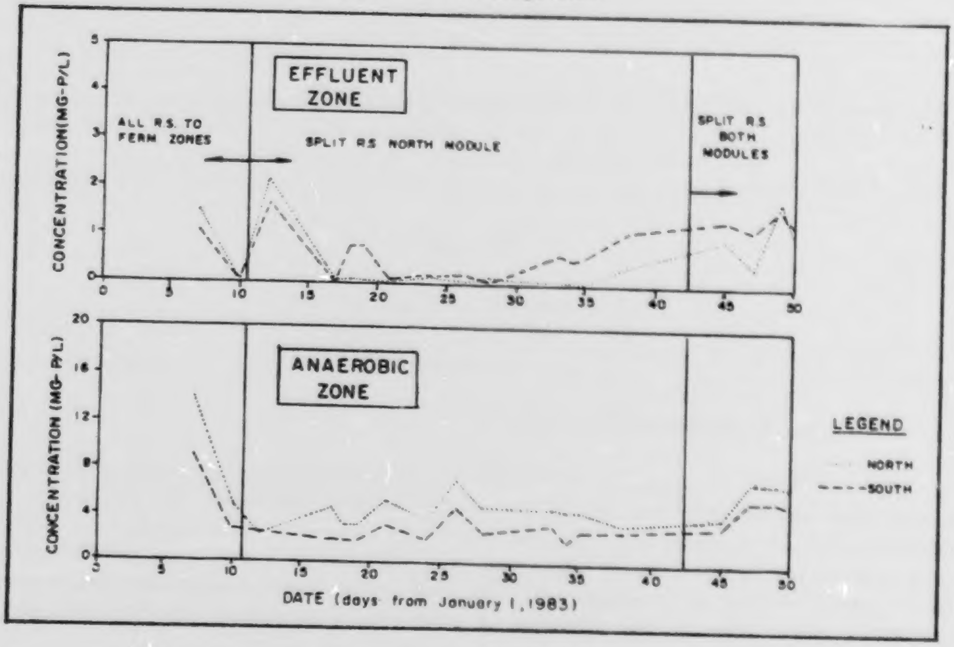

INTERNAL RECYCLE
ANAEROBIC
ANOXIC
AEROBIC
ANOXIC
AEROBIC
CLARIFIER
SPLIT 50%
RETURN SLUDGE

FIGURE 8

Split Return Sludge Test



4. Secondary Phosphorus Release

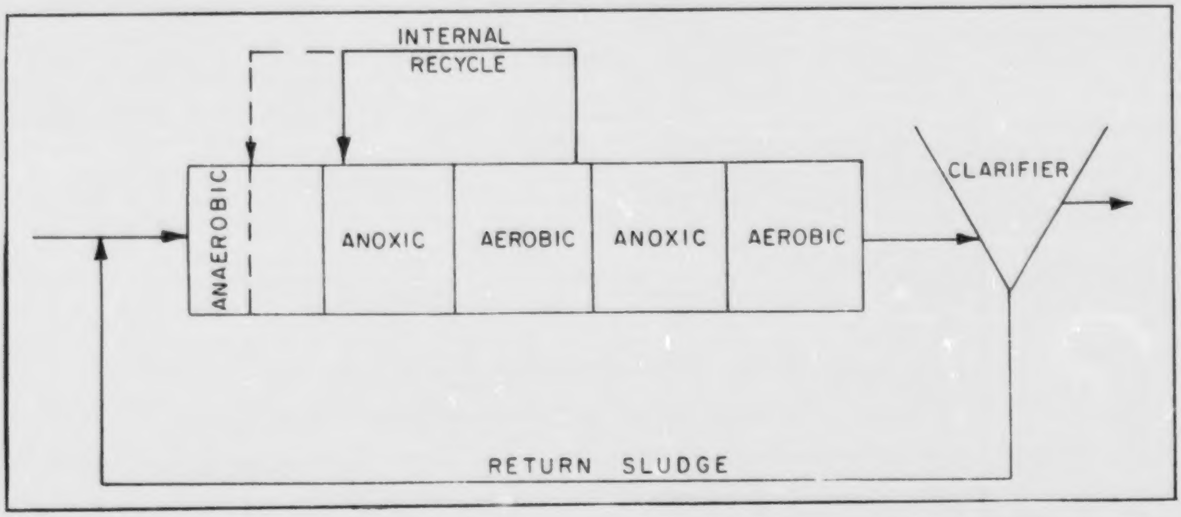
It was noticed that, during the experimental split return sludge test, the mass phosphorus released in the anaerobic zone per mg of activated sludge did increase. This same increase was noticed with the addition of additional volatile fatty acids from the gravity sludge thickener or bagged sodium acetate. The additional release of phosphorus could also be observed in the second anaerobic zone; however, the relationship of higher release of phosphorus and higher phosphorus removals was not apparent. At this time it was postulated there may be more than one type of phosphorus release mechanism occurring in the anaerobic zone and that the secondary release may not be beneficial to subsequent uptake of phosphorus in the aerobic zones.

5. Reduction in Anaerobic Zone

The only available test to check this theory was the addition of temporary recycle of mixed liquor containing nitrates from the end of the aerobic zone back into the second anaerobic zone (Figure 9). This resulted in a shorten anaerobic zone and decreased release of phosphorus. The improvement in phosphorus removal became apparent almost immediately and the effluent phosphorus values dropped by approximately 1 ppm and at times ran less than one part per million.

FIGURE 9

Reduction in Anaerobic Zone

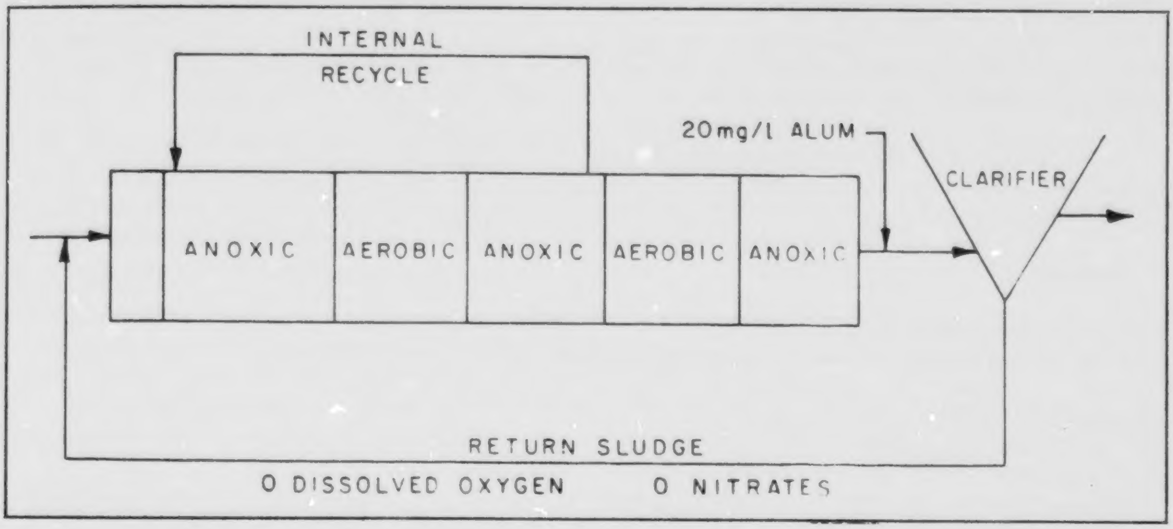


6. Denitrification in Secondary Clarifiers

The denitrification reaction in secondary clarifiers was long ago considered to be a serious operational problem and was to be avoided if at all possible. In the Kelowna plant the concentration of nitrates entering the clarifiers was only 1.0 - 2.0 ppm and the possibility of a problem was considered low enough to give the concept a try. The possible benefit would be to lower the mass of nitrates and dissolved oxygen returning in the return sludge to the anaerobic zone. One of the problems of operating a secondary clarifier without dissolved oxygen in the biological removal system was the possibility of a release of stored phosphorus and one solution was to add a very small dose of alum to precipitate the released phosphorus. With the conversion of the secondary clarifiers to denitrification mode and the addition of 15 – 20 mg/l of alum came the fast reduction of effluent phosphorus and the stabilization of the biological phosphorus removal mechanisms (Figure 10).

FIGURE 10

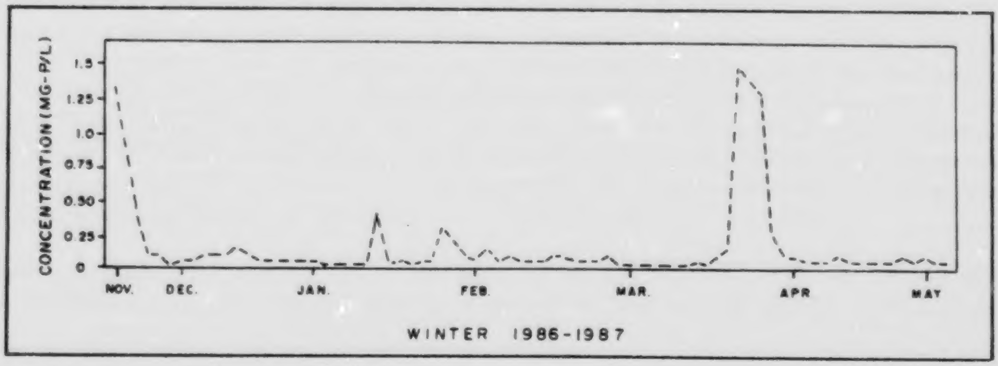
Highest Phosphorus Removal


INTERNAL RECYCLE
20mg/l ALUM
CLARIFIER
ANOXIC
AEROBIC
ANOXIC
AEROBIC
ANOXIC
RETURN SLUDGE
0 DISSOLVED OXYGEN
0 NITRATES

The operation of biological phosphorus removal systems has been well researched around the world and there are some aspects which are still being postulated. However, the Kelowna plant has in the last year demonstrated the ability to consistently achieve very high removal of phosphorus with some new ideas in the process train (Figure 11).

FIGURE 11

Secondary Clarifier Effluent Ortho-P



THE KELOWNA FACILITIES

Sewer System

Raw sewage enters the plant from a seperate storm and sanitary sewer system of relatively short retention time. The sewer system is a combination of gravity fed and pumped drainage areas and the final 2,000 meters is gravity to the plant. The average hydraulic retention time would be less than 4 hours thus generation of rapidly biodegradable compounds in the sewer system through most of the year is not a major factor. Compounding this short hydraulic retention time is the infiltration into the system from sump pumps and leaking pipe joints. The infiltration quantities amount to 40-50% of the entire flow to the plant consequently the raw sewage has an average B.O.D. of 200 - 250 mg/L.

The plant has a design capacity of 22.5 mega/liters per day and present flows range from 16 to 21 ml/day. Daily peak flows reach 300 L/sec. with overnight minimums of 120-130 L/sec.

The City currently operates a separate TRADEWASTE treatment plant for the two major industries and generally only sanitary sewage and small service industries are connected to the main treatment plant.

Headworks

At the plant the raw sewage passes through a commutating device, grit chambers, parshall flume prior to being pumped to the primary clarifiers by variable speed pumps.

Primary Clarifiers

The primary clarifiers are equipped with a scraper mechanism which drags the sludge to the influent end and collects the sludge in a series of hoppers. The primary sludge pump removes this sludge at a rate of 10-12 L/sec. [equivalent to 6-7% of the influent flow] and delivers the sludge to the primary sludge thickener.

The overflow weirs are followed by a water level control gate which submerges the weirs and allows the primary effluent flow to drop approximately 0.5 meters without the addition of any oxygen. This has proved very important as the following zone in the bio-reactor must minimize the addition of both oxygen and nitrate-nitrogen.

Primary Sludge Thickener

Prior to expansion, the former process configuration used the gravity settling tank to thicken the combination of waste activated and primary sludge. The objective was to concentrate the sludge and store it prior to incineration. In the current process configuration sludge is thickened and the tank has much more capacity than required for this purpose. The hydraulic retention time would average 10 hours based on the pumping rate of 10 L/sec. The sludge blanket in this tank is manipulated in the 1.5 to 4.0 meter range by adjusting the daily draw-off rates. This facility has become the major source of rapidly biodegradable compounds for use in the process which will be discussed in this paper.

Flow Equalization

The former activated sludge facility used the aeration tanks with a short hydraulic retention time. With a few modifications it has allowed us to use these tanks as flow equalization vessels with storage of peak flows during the day and flow supplement at minimum flows during the night. The total volume of the tanks amount to only 6% of the ultimate design flow; however, the release of organics during the low flow period greatly assists us in control of dissolved oxygen in the bio-reactor.

Bio-Reactor

The configuration chosen was the five stage phoredex process (BARDENPHO) and it was decided to construct the plant with two identical plants each with approximately 11 mega/litre per day capacity. The layout resulted in the two plants operating completely independent of each other and this allows changes on one side while the other remains as an on-line reference module. Four of the major operational parameters chosen for control were:

1. Retention time by zone
2. Dissolved oxygen control
3. Internal recycle flow control
4. Sludge age control

Retention time control was necessary due to the expected temperature range of 8-25 degrees Celsius and the subsequent effect on reaction rates such as nitrification.

The bio-reactor consist of 21 cells with aeration facilities in 17 of the 21 cells. The stirrers in the zones with aeration facilities were equipped with two speed stirrers which gives the possibility of efficient oxygen transfer at high speed or minimal D.O. input when on low speed (anoxic). Manipulation of the cell configuration results in alteration of the retention time per zone with some of the more likely combinations listed below:

Zone	# of Cells
1. Fermentation	1-3
2. 1st anoxic	2-7
3. 1st aerobic	4-7
4. 2nd anoxic	0-4
5. Re-aeration	1-3

However, as you increase or decrease the number of cells in any one zone you increase or decrease the adjoining zone, thus it has become an exercise in optimization of all the zones.

Dissolved oxygen control was achieved by the monitoring of residual oxygen levels in the 1st aerobic zone and maintaining an average of 2.0 mg/L over the entire zone. The probes feed their information into the process computer which in turn increases or decreases the amount of air to the entire bio-reactor. An additional D.O. control zone was added to the re-aeration cells with a D.O. probe changing the stirrer speed (i.e. from high speed to low speed).

Internal recycle flow control was accomplished by the installation of 3 low head recycle pumps and the ability to restrict the flow from each pump thus giving control from less than 220

L/sec. to 760 l/sec. This gave a possible range of internal recycle/influent ratio of 3:1 up to 12:1.

Sludge age control was achieved by wasting from the re-aeration zone to dissolved air flotation tanks. The manipulation of the daily waste flow totals permits the control of sludge age by hydraulic wasting. Each day the pumping of 5% of the volume of the reactor results in approximately a 20 day sludge age. This of course is based on the assumption that all the sludge is in the bio-reactor which in our case, due to the size of the bio-reactor, is a very close assumption. Wasting from the re-aeration zone eliminated the concern of variable sludge concentration when wasting the clarifier underflow.

Secondary Clarifier

The clarifiers chosen were the rim drive, centre feed, rim draw-off design. The design results in a slow sludge removal mechanism which allows some control over the sludge retention time in the clarifier.

It is important not to leave the sludge in the clarifier too long as it will release the biologically stored phosphorus and also it is important not to return too much dissolved oxygen or nitrates to the bio-reactor's first zone. Th clarifier return sludge pumps were chosen to give up to 100% recycle (return/influent) and yet still maintain flow control. Low head pumps were chosen with magnetic flow meters for flow measurement.

Effluent Filtration and Chlorination

One of the original options for effluent disposal was to re-use the effluent for spray irrigation of food crops rather than disposal of the lake system. This option required effluent filtration and an oversize chlorine contact tank to be added to the facility.

The filters chosen were dual media sand and anthracite with travelling bridges for backwashing. The filters remove the 3-10 mg/L suspended solids from the clarifiers to virtually non-detectable levels. They also remove the total phosphorus and organic nitrogen associated with the solids. This results in approximately 0.3 mg/L total phosphorus and 0.4 mg/L total nitrogen removals. The non-detectable levels of ammonia in the effluent and the addition of chlorine results in breakpoint chlorination with dose rates of 5-8 mg/L. COLIFORM levels in the final effluent have been very difficult to measure and often the result is less than 1 MPN.

Blower System

The blower system was chosen to meet a wide range of air requirements and to minimize the power requirements. Four blowers were used to supply the low flow requirements of 500 L/sec. and also be sequenced up to the full load requirement of 4,000 L/sec. To accomplish this, the centrifugal blowers were each given a flow control system which results in a 50% turn down capability and a subsequent 40% turn down in amperage draw.

The process computer was given full control of blower operations and will start/stop or modulate the output of the blower system to meet the air requirements as measured by the residual dissolve oxygen probes. Average D.O. levels often remain between 1.8 - 2.2 mg/L for greater than 98% of the day.

Dissolved Air Flotation

The dissolved air flotation system was chosen to handle the waste activated sludge for basically two reasons. It would accommodate the theory of hydraulic wasting very easily and would do it with very little input of manpower. The second reason is that the process is totally aerobic thus there would be no re-release of phosphorus.

Computer Control

The desire to limit the use of manual labour for plant operations and to permit the operation of the plant 24 hours/day under stringent operating conditions resulted in the virtual automation of the entire plant.

The Texas Instrument Programmable Logic Controllers handle the operations of the motor control centers. Starting and stopping motors or other contact inputs and outputs are programmed into the sequencers using a video programming unit. Once the sequencer has been programmed, total automatic control is available. Process control is handled by two Fischer and Porter DCI 4000 computers which have the capability of analog logic control. Sequential control using analog input/output and dry contact input/output results in the ability to control and alarm all functions such as blower operations with set point control of dissolved oxygen.

Data management is handled by the third computer system developed by Westhead Associates to communicate with the DCI 4000 system. Every ten minutes 24 hrs/day the DCI dumps up to 256 analog values for storage. This is combined with the operator input for such information as laboratory results to produce a comprehensive reporting program.

The isolation of the three computer functions has resulted in the isolation of major breakdowns to the specific computer involved. The programmable logic controllers have proven the most reliable with several minor failures at start-up and none since. The process computers have had several failures associated with lightening strikes which we are in the process of providing a power supply completely free of electrical spikes.

OPTIMIZATION OF FULL-SCALE GRANULAR ACTIVATED CARBON CONTACTORS AT THE BUFFALO POUND WATER PLANT

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INTRODUCTION

A full scale granular activated carbon post-filtration contactor system was installed at the Buffalo Pound Water Plant in June 1985 as the solution to a long-standing taste and odour problem with the drinking water in the cities of Regina (pop. 175,000) and Moose Jaw (pop. 35,000). The plant operated successfully in 1985 under moderate loadings in removing algae-related odours[1] but a heavy algae bloom in 1986 caused very high odour and geosmin influent levels for a 2-3 week period in August with almost immediate low level breakthrough of grassy, musty odours. Performance for total organic carbon and trihalomethane removal was essentially the same for the two years.

The breakthrough phenomenon was reviewed and it was confirmed that there had been no failure of any treatment component, but rather that the incoming odour levels had exceeded the capacity of the carbon under the current operating conditions. Pilot work was then undertaken to try and priorize the parameters which affected carbon performance including mesh size, flow rates, series versus parallel operation, and backwashing effects. In addition pre-treatment options were considered to reduce the load onto the GAC contactors during peak periods; these included changing or eliminating pre-chlorination and adding powdered activated carbon. It was hoped to optimize all of these factors so as to make the GAC contactors able to cope with peak odour conditions.

BACKGROUND

The major source of water for the two cities is Buffalo Pound Lake, a 30 km (18 mile) long and 1 km (0.6 mile) wide artificial reservoir with a mean depth of 3 m (10 ft.). Extensive blooms of blue-green algae (anabaena, aphanizomenon, and microcystis) occur in summer[2] when the temperature of the water reaches 20-25 deg C. These algae blooms cause a number of problems, primarily in July and August, the most evident to the consumer being the objectionable taste and odour, which has been described as sour, musty, grassy, septic, pigpen, etc.

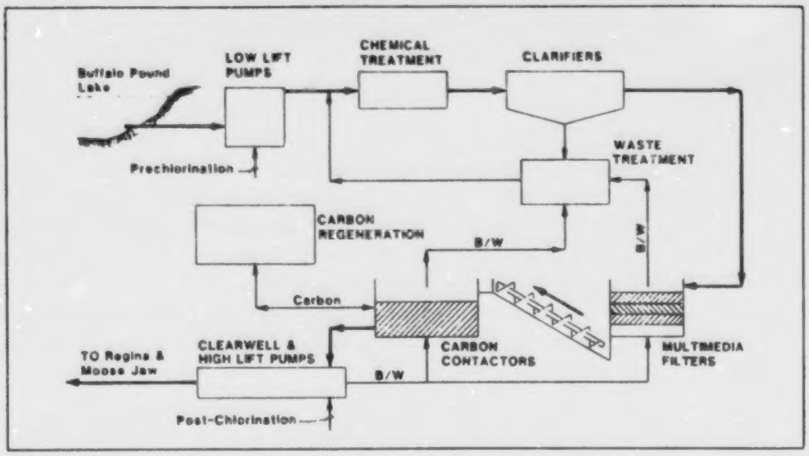
Under average summer conditions, the plant normally produced a finished water with low turbidity (<0.2 NTU), and colour (<5 Pt/Co units), but with high threshold odour numbers (5-15), high total organic carbon (3-8 mg/L C) and appreciable trihalomethanes (70-150 ug/L).

FACILITY DESIGN

Before the GAC addition the treatment plant was of conventional design with pre-chlorination, alum coagulation/flocculation with polymer aids, settling in upflow reactor-type clarifiers equipped with 60 degree tube settlers, pH correction, and mixed-media filtration. Installation of the GAC system in June of 1985 (Figure 1) now gives additional reductions in odour, organic carbon and trihalomethanes. Plant flows are 54-136 dam^3/day (12-30 MIGD) and water is delivered from the plant clearwell through a 58 km (36 mile) pipeline to Regina and a 20 km (12 mile) pipeline to Moose Jaw.

FIGURE 1

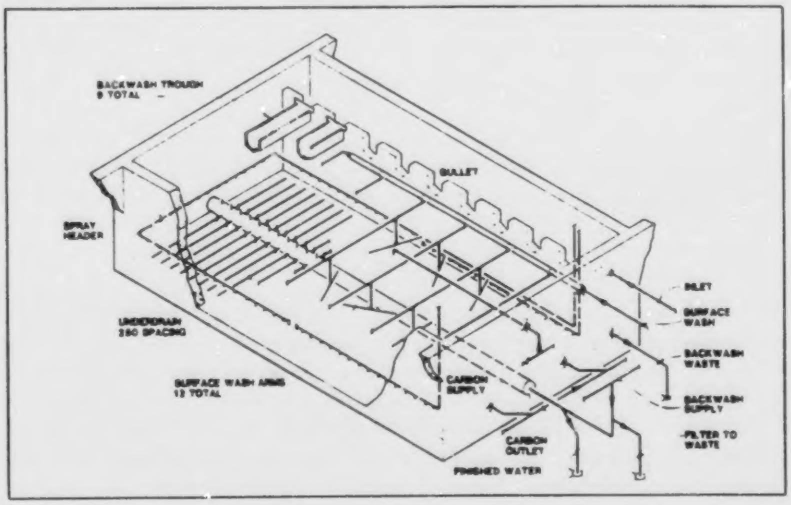
Plant Schematic


LOW LIFT PUMPS
CHEMICAL TREATMENT
CLARIFIERS
Buffalo Pound Lake
Prechlorination
WASTE TREATMENT
CARBON REGENERATION
B/W
B/W
Carbon
CARBON CONTACTORS
MULTIMEDIA FILTERS
TO Regina & Moose Jaw
CLEARWELL & HIGH LIFT PUMPS
B/W
Post-Chlorination

The new GAC contactors^3 are basically of concrete filter box design (Figure 2) with 58.5 m^2 (630 ft^2) of surface area, 3 m (10 ft) depth of carbon and another 2.1 m (7 ft) of water depth from the carbon to the backwash troughs. The underdrains consisted of plastic pipe laterals wound with 0.5 mm wedge wire. From the contactors the water passes into a clearwell area where it is post-chlorinated for distribution system disinfection. The 8 beds each contain 172 m^3 (6074 ft^3) of carbon and are designed for a minimum empty bed contact time (EBCT) of 15 min (maximum flow rates of 12.1 m/hr, 4.12 igpm/ft^2). Total amount of carbon in use is 689,000 Kg (1,520,000 lbs).

FIGURE 2

Contactor Detail


BACKWASH TROUGH
8 TOTAL
GULLET
SPRAY HEADER
INLET
SURFACE WASH
UNDERDRAIN
250 SPACING
BACKWASH WASTE
CARBON SUPPLY
SURFACE WASH ARMS
12 TOTAL
BACKWASH SUPPLY
FILTER TO WASTE
CARBON OUTLET
FINISHED WATER

One of the contactors was equipped with stainless steel sample ports at 30 cm (1 foot) intervals down the side wall over the depth of the carbon. These ports were screened and extended 45 cm (17.7 in) into the carbon.

Carbon Regeneration

The carbon is regenerated at an onsite facility on a yearly basis. The furnace is a Westvaco two-stage, gas-fired, fluid bed system[4], which has a capacity of 6350 Kg/day (14,000 lbs/day), deliberately oversized to allow for future water plant expansion plus the possibility of more frequent than anticipated regeneration. It is capable of regenerating the entire present inventory in 3.5 to 4 months (1 contactor every 13-16 days).

ANALYTICAL METHODOLOGY

Threshold odour numbers were measured using Standard Methods[5] but were qualified by adding one or two adjectives to describe the odour.

Trihalomethanes were monitored[6] on a gas chromatograph (Hewlett-Packard 5840A) equipped with an electron capture detector. A 5-ft Chromosorb 101 packed column (glass, 0.25 in diam.) was used with direct aqueous injection (5 ul samples). Geosmin analyses were carried out on a gas chromatograph (Hewlett-Packard 5730A) equipped with a flame ionization detector and a capillary column (30 m DB-1). A 2-litre sample was extracted with two 5 ml aliquots of pentane and concentrated to 0.5 ml for injection. This method[7] was used to monitor contaminants in the C6 to C35 range, but was primarily set up to look for geosmin and methylisoborneol.

Total organic carbon was monitored either by carbon analyzer (Beckman 915-B IR model) or by UV absorption at 254 nm on a 2-nm bandwidth spectrophotometer using 10-cm cells. The precision of the UV method (+/- 0.1 mg/L) was better than the IR Analyzer (+/- 0.5 mg/L) but the UV method could only be used for filtered samples (DOC).

1985/1986 RESULTS - FULL SCALE

The contactors were monitored continuously for operational problems, flow, turbidity, and headloss, daily for pH, chlorine residuals and standard plate counts, and weekly or better for TOC, trihalomethanes, odour, particle counts, geosmin, phosphate and algae cells. In addition raw and finished water samples were monitored regularly for all major ions, trace nutrients, bacteria, metals, organics and physical parameters. Wavefront progressions for odour, TOC and TTHMs were followed by using the 0.3 m (1-ft) interval ports. The contactors were each backwashed at 8 day intervals in 1985 and 16 day intervals in 1986 primarily to control bacteria levels.

The GAC contactors were operated for 135 days in 1985; this consisted of a 10-day commissioning period in June (10th-19th) and then continuous operation from July 3rd to November 4th. The carbon successfully removed odour and reduced TOC and TTHMs over the 4.5 month period. By November odour had broken through at low levels and TOC and TTHM removals were down to 30-40%. Effective total exhaustion for TOC and TTHM (effluent consistently at 95% or more of influent) was extrapolated at around 170-200 days.

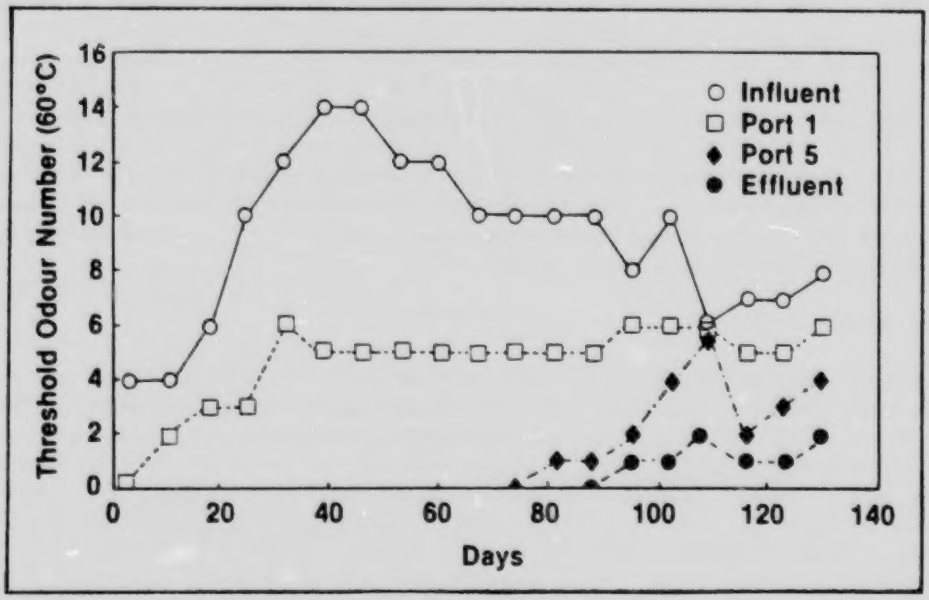
The contactors were on for 139 days in 1986 (June 20-Nov 6) and the peak load occurred at days 45-70 (mid-August). Whereas the algae blooms in 1985 were of low to moderate severity, the algae bloom in 1986 was severe (chlorophyll a peaked at 66 ug/L) and it produced a very strong odour peak (threshold odour numbers of 45-55) which the carbon could not handle completely, with low-level odours passing the GAC within a week of peak loads. During the odour peak 0.60 m (2 ft) of extra virgin carbon was added to one contactor as an emergency measure. After the peak passed the contactors continued to leak low levels of odour (TON) and geosmin. Incoming levels and removals of TOC and TTHMs however were essentially the same for both years and gave no indication of any difference in water quality or in treatment.

Odour

In 1985 Raw lake-water odours of 10 to 30 TON (musty, sour) were reduced to 4 to 14 by conventional treatment and the GAC then eliminated this odour for 90 days (Figure 3) after which low-level odours of 1 to 2 TON units of a slightly musty odour, resulting in few complaints, were found in the effluents. Odour breakthrough at the 1.52 m (5-ft) level occurred at 75 days, so the bottom 1.52 m (5-ft) layer did not appear to be as effective in removing odour as the top of the bed.

FIGURE 3

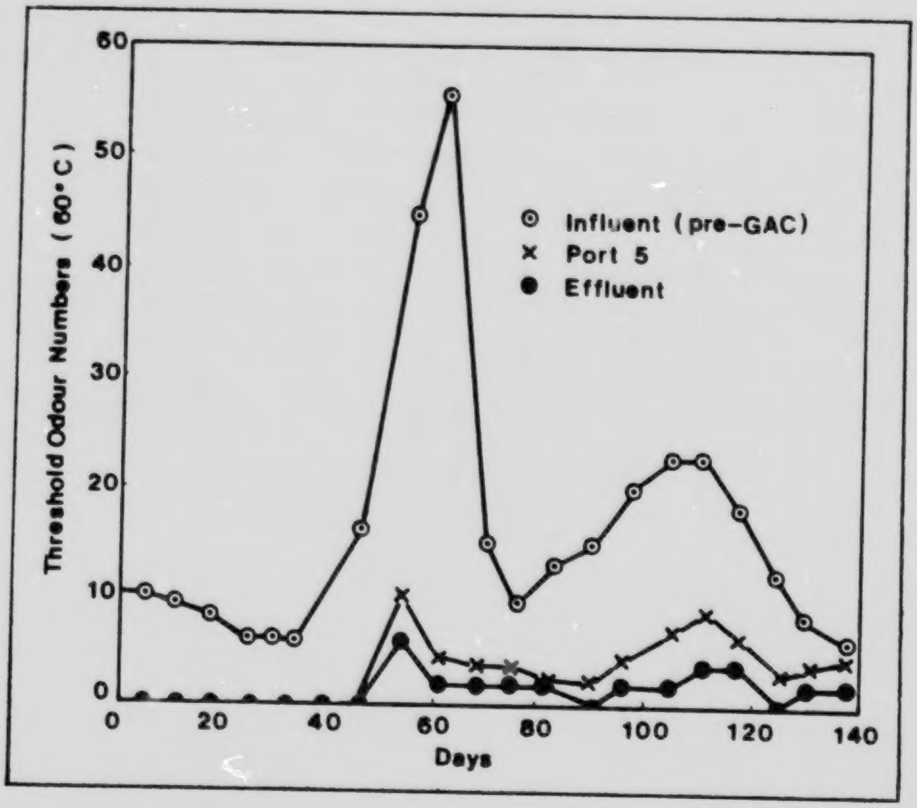
Odour Breakthrough 1985



In 1986 however odour levels peaked at 55 TON going on to the GAC and for two weeks conventional treatment did little to reduce odour levels. Grassy, musty odours broke through the carbon within a few days of the peak loading, reaching a high of 5 TON and quickly dropping to TON values of 1 or 2 (Figure 4). The carbon was removing most of the incoming odour but enough of it broke through to give noticeable grassy, musty odours in the finished water. The addition of the extra 0.60 m (2 ft) of carbon to the ported contactor gave a small improvement to effluent levels but was not enough to stop continued breakthrough of low level odours.

FIGURE 4

Odours 1986

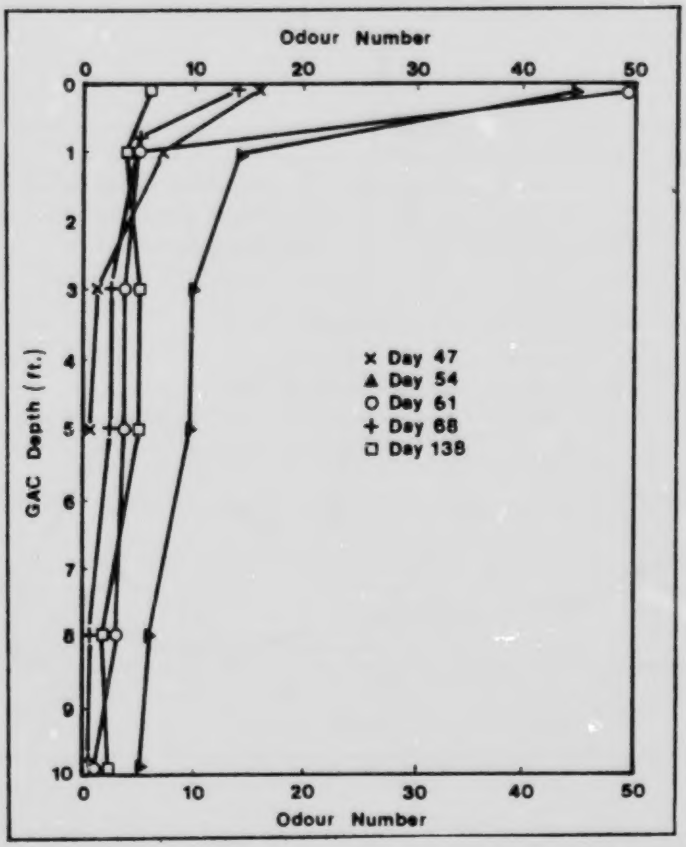


Wavefront progressions for odour (Figure 5) were found somewhat surprisingly to be very similar to the wavefronts for TOC or TTHM, being very broad and steep and with little removal occurring in the bottom half of the bed. This means that if the influent levels are high enough then breakthrough of detectable levels is inevitable.

We had a minor odour problem in both years (additional TON of 2 to 4) due to chlorinous odours formed after post-chlorination of the contactor effluents. This odour was a nitrogen trichloride, swimming pool type odour which was evident within two weeks of start-up and appeared to be caused by reaction of the free chlorine with organic nitrogen compounds which were breaking through the carbon. Public response indicated that while people were noticing the chlorinous odour they found it much less objectionable than the algae odours. An air bubble diffuser system was installed in the clearwell in late August 1986 and this has reduced but not eliminated the chlorinous odours (it also serves as a pH adjusting agent by reducing dissolved carbon dioxide).

FIGURE 5

Odour Wavefront Progression 1986



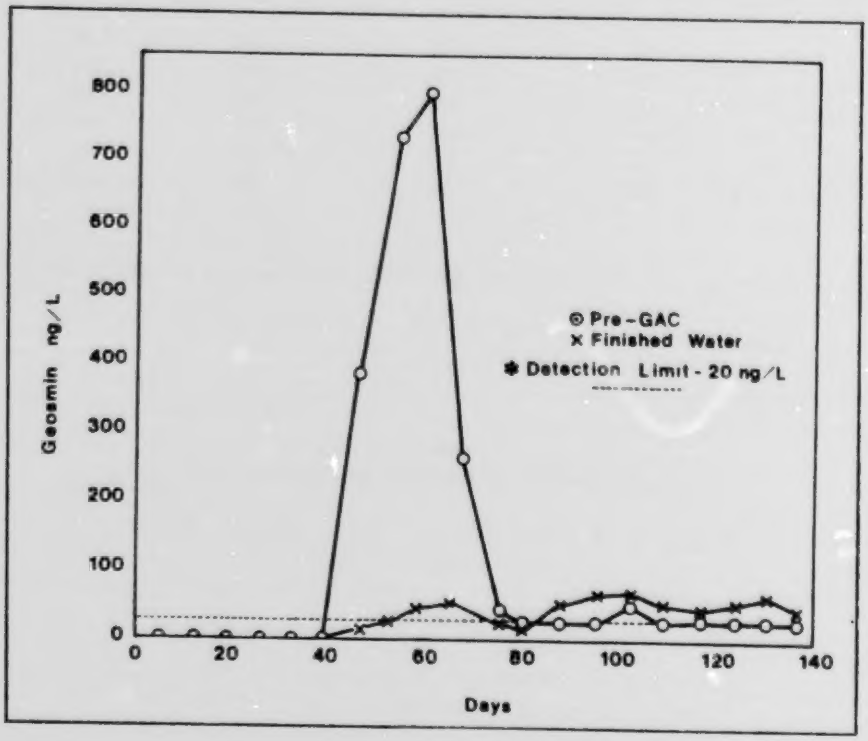
Geosmin

Geosmin was only detectable for 4 weeks in 1985 in late July and early August in the pre-GAC water at levels of 20-230 ng/L while odours were present throughout and when we did observe an odour breakthrough from the GAC no geosmin was detected, so the carbon successfully removed all incoming geosmin. Likewise although low-level odours were found in the distribution systems no geosmin was found. No methylisoborneol was detected in any sample.

In 1986 by contrast geosmin increased from non-detectable (<20 ng/L) in July to a high of 800 ng/L (days 45-70) and it broke through the carbon within a few days of the peak load (Figure 6). Effluent levels reached a maximum of 50 ng/L and continued to bleed from the GAC till day 130. The GAC was removing in excess of 90% of the influent peak of geosmin but it did not remove it completely.

FIGURE 6

Geosmin Data 1986



Cumulative loadings calculated for geosmin gave 2.4 ug/g of GAC at breakthrough and 2.7 ug/g of GAC at day 130. Soxhlet extractions performed on GAC core samples after the GAC was shut off gave recoveries of 4 ug/g in the top of the bed and 2 ug/g 2.1 m (7 ft) down in the bed. It thus appears that loadings were as calculated and that there was no biodegradation of the geosmin in the bed.

Data from 1985 to 1986 had shown that pre-chlorination appeared to be increasing the amounts of geosmin from the raw water through the plant to the pre-GAC water (Table 1). A set of experiments[8] in 1986 showed that increasing pre-chlorination increased geosmin levels whereas chlorination following 0.45 um filtration to remove algae cells gave the same geosmin levels as were found in the raw water (Table 2). It appears that the chlorine is releasing geosmin from the algae cells. It was noted at the same time that pre-chlorination was reducing sour, septic odours but increasing musty odours.

TABLE 1
Geosmin Levels* as a Function of Degree of Water Treatment

Date [1986]	Raw Water^{+}	Chlorinated Raw++	Pre GAC##	Post GAC**
July 31	<20	<20	<20	<20
August 5	32	203	389	<20
August 11	24	945	722	<20
August 18	164	615	798	36
August 25	67	177	286	43
September 8	<20	<20	<20	<20

* In nanograms per liter; geosmin detection limit by this method is about 20 ng/L
+ Untreated lake water at plant intake
++ Raw water treated with 3-4 mg chlorine/L for 30-60 minutes
## Chlorinated, clarified, filtered water
** After passing GAC contactors

TABLE 2
Geosmin Levels as Function of Chlorine Dose

Samples*	Geosmin [ng/L]
Raw Water	164
Raw + 1 mg chlorine/L	458
Raw + 3 mg chlorine/L	1,233
Raw + 5 mg chlorine/L	906
Filtered raw + 3 mg chlorine/L	142

* Prechlorine demand [60 minutes] on August 22, 1986, was 3 mg/L; water temperature was 22°C.

Trihalomethanes

Trihalomethane levels going on to the carbon were moderate in 1985, peaking at 70 ug/L and dropping to 40 ug/L by November. Breakthrough came early with low levels appearing at the effluent within days of start-up [Figure 7]. Levels generally decreased with bed depth, but adjacent segments in the lower part of the bed often showed little difference and occasionally data appeared to indicate some desorption.

Results for 1986 showed very similar trends with influent levels of 40-60 ug/L, early breakthrough and removals dropping to 40% by day 130. An additional 0.60 m [2 ft] of carbon was added to the contactor with the ports at day 59 and the results show a limited improvement for 20-30 days [Figure 8].

We compared the performance of virgin carbon with regenerated CECA and regenerated Calgon carbon to confirm that our regeneration system was providing us with good quality activated carbon [Figure 9]. Results show that the different carbons are essentially the same for TTHM removal within experimental error.

FIGURE 7

TTHM Breakthrough 1985

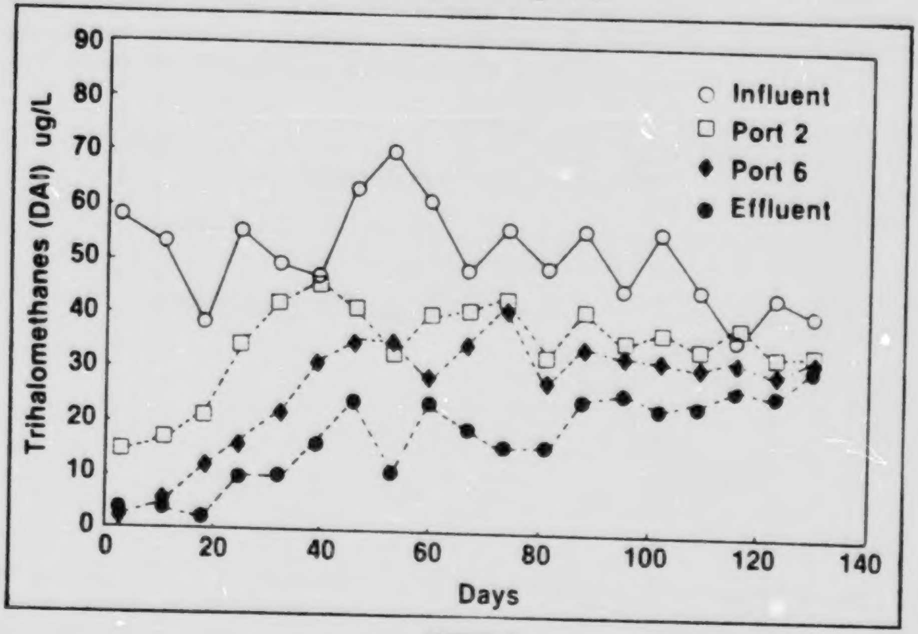


FIGURE 8

TTHM Breakthrough 1986

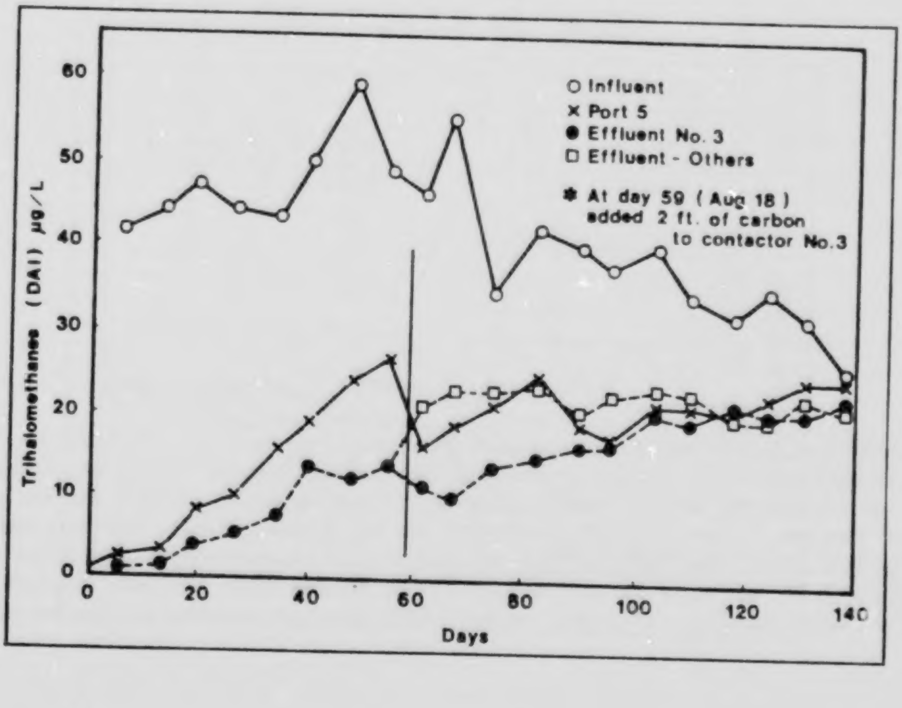
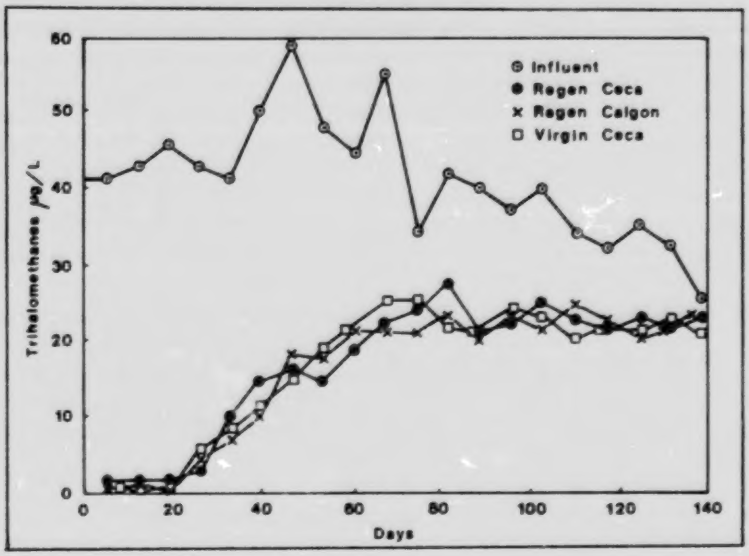


FIGURE 9

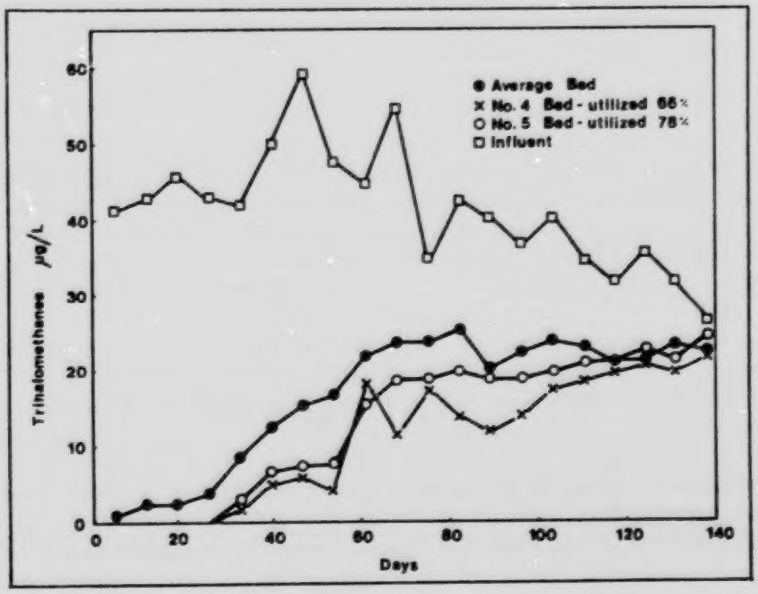
TTHM Comparison of CECA/Calgon/Virgin Carbons 1988



Performance was also compared on 2 contactors which had somewhat lower usage (bed volumes 66% and 78%) than the others (they were taken out of service when water demands dropped below 90% of full flow (1 out) or 80% (2 out). Figure 10 shows that with lower usage there was some improvement but it was not in proportion to amount of use.

FIGURE 10

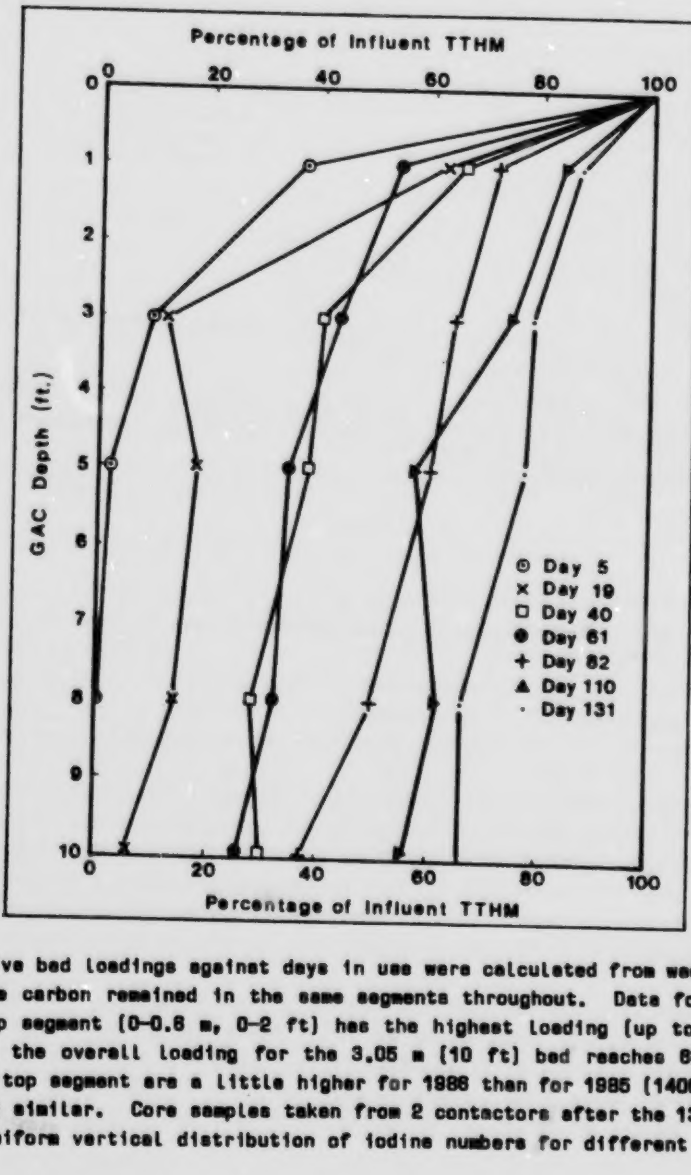
TTHM Comparison of Contractors 1988



Wavefront progression through the bed for both years was very similar and shows (Figure 11) that the front is very broad extending over the whole depth of the contactor with best removals being in the top of the bed whereas the bottom 1.52 m (5 feet) shows only marginal removals from segment to segment. Lines occasionally have a negative slope indicating desorption, TTHM formation or bed mixing on backwash. The addition of the extra 0.60 m (2 ft) to the ported contactor changed the wavefront progressions only slightly.

FIGURE 11

TTHM Wavefront Progression 1986



Apparent cumulative bed loadings against days in use were calculated from weekly values at each segment as if the carbon remained in the same segments throughout. Data for 1986 (Figure 12) show that the top segment (0-0.6 m, 0-2 ft) has the highest loading (up to 1800 ug TTHM/g of carbon) and that the overall loading for the 3.05 m (10 ft) bed reaches 600 ug/g of carbon. Loadings for the top segment are a little higher for 1986 than for 1985 (1400 ug/g) but overall loadings are very similar. Core samples taken from 2 contactors after the 138 days use in 1986 show an almost uniform vertical distribution of iodine numbers for different segments (Table 3) which does not agree with apparent cumulative loadings. This implies a degree of vertical mixing of the bed on backwash, which could be the cause of the broad wavefronts and premature breakthroughs. This is currently being investigated in in-house pilot studies.

FIGURE 12

TTHM Cumulative Bed Loadings 1986

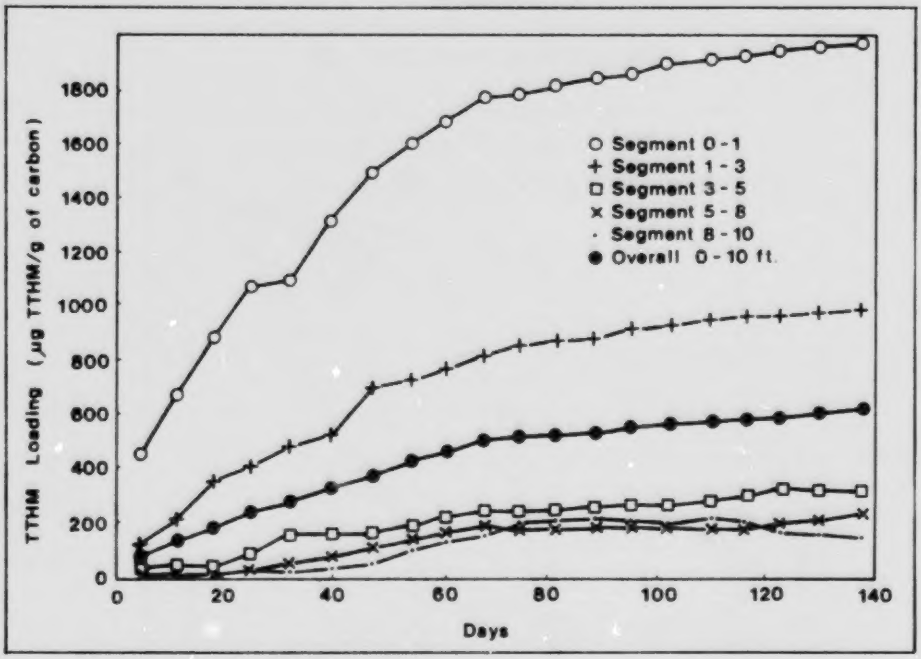


TABLE 3

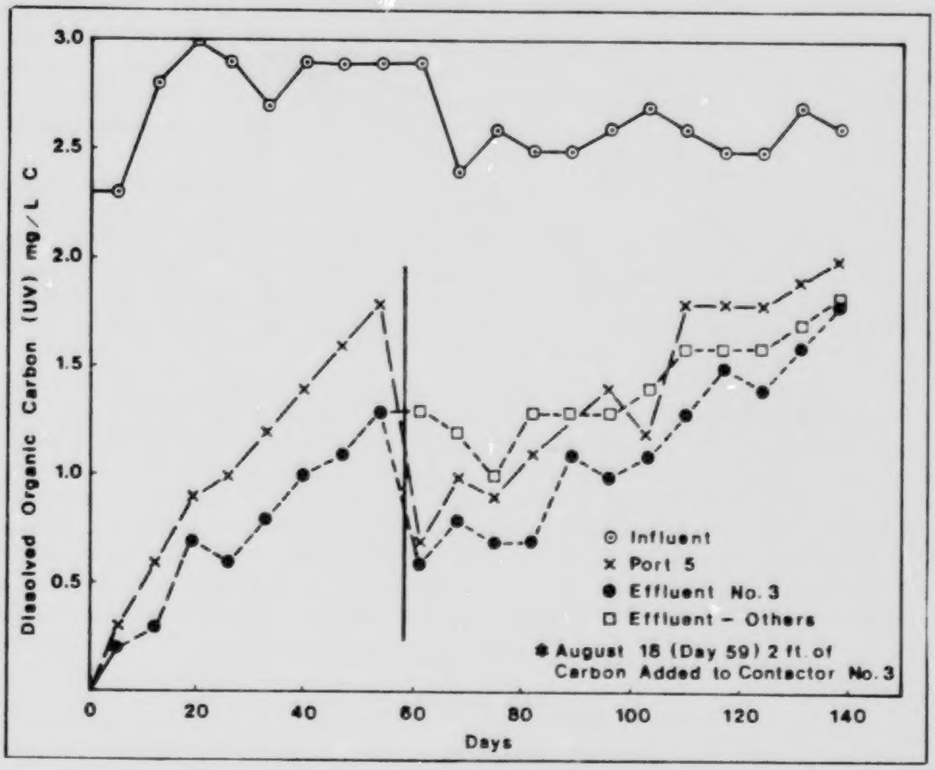
Iodine Numbers from Spent GAC Contactor Core Samples

Depth Segment	Iodine Numbers mg/g Mean	Max	Min
0 - 0.75 m	758	846	650
0.75 - 1.50 m	749	830	617
1.50 - 2.25 m	745	822	635
2.25 - 3.00 m	750	845	653

Dissolved Organic Carbon

Levels of Raw TOC were moderate in 1985 and 1986 at around 6 mg/L C. Conventional treatment reduced this to 2.5-3.5 mg/L C going on to the GAC. Breakthrough came early (Figure 13) with detectable levels in the effluent after 10 days. Levels generally decreased with bed depth and effluent levels remained below 1 mg/1 C for 50 days and below 2 mg/L for 30 days. The addition of the extra 0.60 m (2 ft) of carbon to the ported contactor did show an improvement for up to 80 days.

FIGURE 13
DOC Breakthrough 1986



Wavefront progression of DOC through the bed is similar for both years (Figure 14) and shows the same broad fronts as in the TTHM case with removal occuring over the whole depth of the contactor, breakthrough very early, and the top of the bed being more effective than the bottom. However in contrast to the TTHM situation the DOC fronts have a more positive slope with little indication of negative slopes (desorption). The addition of the extra 0.60 m (2 ft) of carbon had a noticeable effect on the wavefront, pushing it back to from day 59 to an effective day 20.

Apparent cumulative bed loadings (mg DOC/ g of carbon) (Figure 15) for 1986, calculated as if the carbon remained in the same segments throughout, show that the highest loading is on the top 0.91 m (3-ft) segment (6% loading after 130 days) and that loadings decrease with bed depth. The overall bed loading increases gradually to 3.5% at 130 days. Results are essentially the same as for 1985. Once again Iodine number data from core samples show that there is almost uniform vertical distribution within the bed, implying that there is extensive vertical mixing on backwash.

FIGURE 14

DOC Wavefront Progression 1986

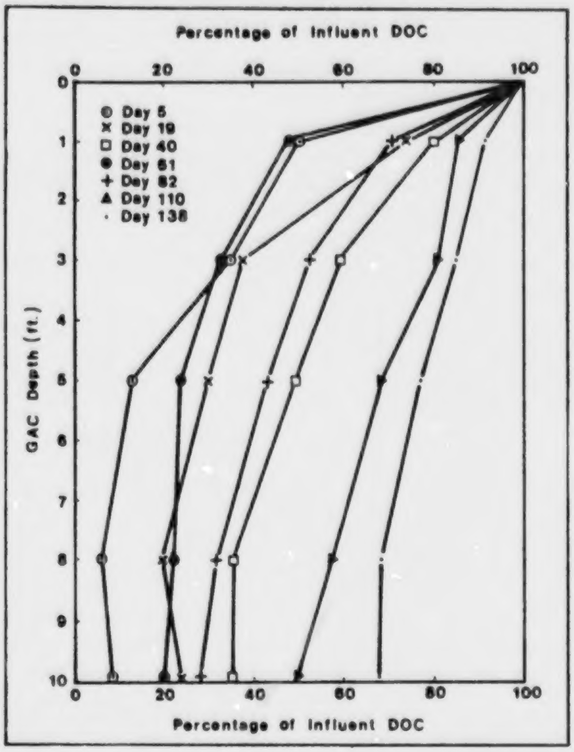
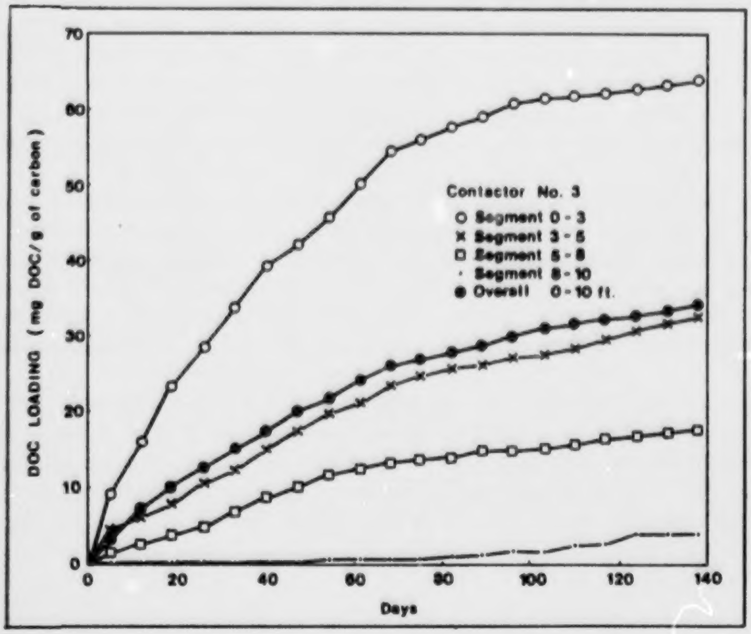


FIGURE 15

DOC Cumulative Bed Loadings 1986



Bed Stratification

Backwashing 3 m (10 ft) of carbon produced stratification in the contactor beds with the most noticeable differentiation being in the top 10% of the bed. The top 0.3 m (1 ft) is essentially 16x30 mesh material (mean size ~0.8 mm) with the top few inches being very fine. Lower layers are generally closer to 8x16 mesh (mean size ~1.5 mm). The gradation in mesh size appears to follow the wavefront progression profile in character and may be one of the reasons that removals are better in the top segments of the bed.

Carbon Regeneration System

The Westvaco fluid furnace was in operation by January 1986 and fulfilled its 24-hour performance specification of 14,000 lbs/day of regenerated carbon at 95% or better of original iodine number and less than 5% smaller than 30 mesh. The first regeneration of the carbon took place from January 10th to May 5th, 1986 and the second from November 9th, 1986 to April 1st, 1987.

Our first year of data on regeneration indicates bed-to-bed losses to be 6.5%, and the second season gave 7.5%, somewhat better than anticipated. Overall losses included scalping 2.5 cm of fines off the top of the backwashed contactors after the carbon had returned from regeneration. Carbon could be produced at required specifications with apparent densities and iodine numbers being similar to virgin material (Table 4) although there was a spread in values (+/- 4%). Sieve analyses on the spent and regenerated material showed that the mean size decreased on average by 0.19 mm (range from 0.1 to 0.4 mm) and that the sieve fraction finer than 30 mesh averaged 5.3% (range from less than 3% up to 10%) compared to the virgin specifications call for less than 5%.

TABLE 4

Carbon Regeneration Data 1986-1987

	Spent	Regenerated	Virgin
Calgon Carbon (F300)			
Apparent Density g/ml	.558	.519	.53
Iodine Number mg/g	741	940	972
Effective Size mm	.88	.76	.88
Mean Size mm	1.47	1.24	1.54
% Finer than 30 mesh	1.1	3.2	1.8
CECA Carbon (Cecarbon 830)			
Apparent Density g/ml	.527	.494	.488
Iodine Number mg/g	798	998	1,013
Effective Size mm	.89	.81	.88
Mean Size mm	1.48	1.31	1.45
% Finer than 30 mesh	.7	1.9	.9

Summary of 1985/1986 Full-Scale Data

The new GAC contactor system fulfilled its main purpose in eliminating algae tastes and odours in 1985 and provided additional benefits of significant reductions in TOC, trihalomethanes and distribution system bacteria levels. It also provided a blanket protection in case of contaminant spills into the water source. Public acceptance of the resulting drinking water was generally very favourable.

In 1986 however a heavy algae bloom caused low level breakthrough of offensive odours which was enough to alert the public and result in numerous complaints about the "algae" odour. Even after the peak load passed low level odours continued to be released from the carbon. The addition of 0.60 m (2 ft) of extra carbon to the top of one contactor after the breakthrough helped only slightly and did not stop continued low-level odour breakthrough. Without the carbon in place odour levels in the cities would certainly have been severe during the peak bloom period.

The main problem appears to be the broad wavefront which exists for all measured parameters with resulting premature breakthrough at low levels before the carbon in the upper part of the bed is exhausted. The most likely explanation of the broad wavefronts is the disruption of the wavefront caused by mixing of the carbon on backwashing[9,10] with resulting decreases in removal efficiencies, but other possible factors are disruption of equilibrium in the bed by sudden massive increases in influent levels or pre-loading of the carbon by TOC such that the capacity for odour molecules is much reduced from expected values. Removals are also affected by particle size and contact time, and unfortunately our worst algae periods usually coincide with our maximum flow rates and shortest contact times. It is difficult to predict parameter breakthrough and exhaustion times since removals are not linear with bed depth.

Loading of geosmin on the GAC was calculated from 1986 data to be 2.4 ug/g of GAC at breakthrough which is somewhat lower than expected. Snoeyink[11] produced an isotherm for geosmin which can be extrapolated to give loadings of 200 ug/g at equilibrium concentrations of 20 ng/L in distilled water and 8 ug/g in water with 10 mg/L of humics. Isotherm data from Lalezary[12] shows loadings of 150 ug/g at equilibrium concentrations of 20 ng/L of geosmin for both distilled water and a 10 mg/L humic solution. At higher equilibrium concentrations of geosmin expected loadings are higher (400-4000 ug/g at 500 ng/L). The organics in our water may be interfering more with geosmin adsorption than do the humics and so reducing maximum loadings to very low levels.

PILOT WORK 1986-1987

Effect of Mesh Size - Pilot GAC Columns

Three 0.15 m (6 in) diameter 2.4 m (8 ft) high columns were filled with 1.52 m (5 ft) of 8x30, 12x40 and 20x50 carbons (Filtrasorb). Columns were run on filtered water at 1 US gpm (12.2 m/hr, 5 US gpm/sqft) and were backwashed daily, primarily to prevent headloss buildup on the finest mesh size column. Influent, 0.30 m (1 ft) depth ports and effluent samples were taken and analysed for TOC, TTHM and odour. Results showed that although there was a noticeable effect at the 0.30 m (1 ft) depth initially (the finer the mesh the better the removals), the effluent values were very similar, and after a few weeks results tended towards the same levels. The effects of backwashing appeared to be gradually minimising the effect of particle size.

Further pilot work is being carried out on the effects of other parameters on GAC carbon performance including:

Effect of Flow Rate - columns run at 23.5, 11.7 and 5.9 m/hr (8, 4 and 2 gpm/ft^2).

Effect of Backwashing Frequency - backwash at intervals of 1, 7 and 30 days.

Effect of Backwashing Rate - backwash at rates of 14.7, 29.4 and 44.0 m/hr (5, 10 and 15 gpm/ft^2).

Effect of two 1.52 m (5 ft) Series Columns versus one 3.0 m (10 ft) column.

Powdered Activated Carbon

Bench and pilot tests showed the possibility of coping with severe peak loads by adding powdered activated carbon ahead of clarification or ahead of filtration to reduce odour and TOC by 10-50% and thus prolong the useful life of the granular carbon. This option is only considered viable if the peak periods are of short term duration (2-6 weeks). Doses of 20 ppm PAC could remove 15% of TOC, at a cost of $24/ML ($110/MIG), and doses of 50 ppm PAC could remove 40% of TOC at a cost of $60/ML ($275/MIG). A feed system has been set up to try this option this summer (1987).

Pre-Chlorination Deletion

Previous work has shown that pre-chlorination appears to increase geosmin levels by as much as 5 fold over raw water levels so alternative means of pre-oxidation had to be examined. Intermediate stage chlorination, chloramines and potassium permanganate were considered as feasible alternatives and bench tests were carried out to determine the optimum combination to provide low geosmin levels while still being able to remove the algae cells in clarification/filtration and oxidise sour, septic odours. Results showed that potassium permanganate as pre-oxidant followed by chlorination after clarification gave enough benefits to outweigh possible disadvantages. This will be tried on full scale this summer, probably in conjunction with powdered carbon feed ahead of clarification.

FUTURE WORK

We hope to be able to optimize GAC contactor performance by finding optimum backwash rates and frequency, by determining the effects of particle size, flow rate and modes of operation. In addition cost-effective pre-treatment will be used to diminish the effects of high peak loads and thus extend the useful life of the carbon. Unfortunately we have to be able to predict episodes of high loading and odour breakthrough and feed the required chemicals before the problem begins since our data indicate that once it has occurred it is unlikely that we will be able to stop it completely.

A few more years of carbon use will likely be required before we can find the optimum combination of treatment processes. An 18-month study[13] is underway using measured data from the contactors to computer-model the performance to help make meaningful predictions for use under different water conditions.

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AN OVERVIEW OF THE INSTALLATION CODE FOR DIGESTER GAS SYSTEMS

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INTRODUCTION

For many years digester gas has been utilized instead of being wasted in the facilities where that gas is manufactured. These facilities are predominantly wastewater treatment plants or water pollution control plants.

With the increased need for control of water pollution there is an increased need for control of the digester gas utilization.

It is therefore timely for this presentation, which is intended to give you an overview of how the safe utilization of digester gas can be controlled.

In this presentation I will use the theme of this convention "Working Together - Key to a Better Future" and relate to:

1. the past;
2. the present;
3. the future;

of regulating the utilization of digester gas.

THE PAST

Up to the time of the compiling of this presentation, there has been no legislated regulations for the utilization of digester gas. However, the C.G.A. - B105 Code has been referenced in contracts as a document for the installation of digester gas systems in water pollution control plants.

With the B105 Code we have come a long way and we certainly have a long way to go but my first question before we get carried away with where we are going is "do we always consider the cost along the way?"

Many people recall Mr. Trudeau saying, "You can have anything you want as long as you are prepared to pay for it."

Well that's a fact because no matter what it costs to further regulate the safe utilization of digester gas the cost is ours indirectly and we, you and I, will be paying for it through taxes.

The conception of the B105 Code, then, an interim safety requirement, took place in the mid 1960's as a result of several branches of the Ontario government and other related organizations being involved in control of water pollution.

The plants set up to control the water pollution were using anaerobic digesters, which produced a gas, and as this gas had a high concentration of methane it was quickly recognized as an energizing fuel.

It was then determined this fuel could be used in these plants to energize the gas fired: heating and process boilers, the industrial engines and sludge incinerators. Because the plants varied in size, the production of methane, more commonly called sludge gas, varied also, so it was necessary to have these appliances set up with dual fuel burners using: natural gas, propane or oil as the standby fuel.

This was progress and of much interest to energy conservationists but utilizing digester gas presented us with four main problems.

It is flammable.

It is toxic.

It is a wet gas.

It is a corrosive gas.

It was this corrosive nature of the gas that resulted in the regulatory bodies deciding that the existing installation code for natural gas did not provide for the safe utilization of digester gas.

So a team of personnel from the Ontario Water Resources Commission and the Ontario Department of Energy Management compiled "The Safety Regulations for Plant Operators" in 1968, followed by the "Interim Safety Requirement I.S.R. 105" in September 1969. The next publication was the O.W.R.C. "Safety Regulations for Wastewater Treatment Plants" in 1971. Later, the I.S.R. 105 was revised by the Ministry of Consumer and Commercial Relations, the Ministry of Environment, the Ministry of Labour and the Ministry of Health when the I.S.R. April 1973 was published.

It was at that time when the Ontario M.C.C.R. and the M.O.E. enlisted the Canadian Gas Association as an agency to review drawings and inspect systems utilizing digester gas.

C.G.A. was also requested to take over the I.S.R. 105 and get it adopted as a Canadian Standard or Code. This resulted in setting up a C.G.A. Standards Committee, which first established the C.G.A. Safety Requirement S.R. 105-77, then after many meetings of working together, published the Canadian National Code Can1-B105-M81 Installation Code for Digester Gas Systems.

The B105 Code Committee is made up of members from the M.C.C.R., M.O.E., M.E.A., Consulting Engineers, Manufacturers and Inspection Personnel from other provincial governments across Canada.

In a recent survey conducted by a committee member from Gore and Storrie, it was established that most of the provincial governments were using the B105 Code as a Reference Document.

That is "How Far We Have Come".

THE PRESENT

At present our progress is almost at a standstill because the two Ontario Ministries: M.C.C.R. and M.O.E. are still in debate to decide who will accept the responsibility and regulate the utilization of digester gas by legislating the C.G.A. B105-M87 Code. The M87 edition of this Code is the result of the Code Committee working together for six years on revisions to the M81 edition.

This installation code is one of the more than 90 Standards and Codes sponsored by the Canadian Gas Association.

THE FUTURE

You know how far we have come, and I have indicated we are almost at a standstill, but my positive attitude tells me to look ahead to that better future. I still believe we are about to take a giant step forward which will give us "The Key to a Better Future".

The Ontario M.C.C.R. and M.O.E. are meeting to decide when ministry has to accept the responsibility of setting up an Ontario Regulation which will legislate the C.G.A.-B105-M87. This means that the authority having jurisdiction, referred to in several clauses in the B105 Code, will be one of those ministries in Ontario. We believe this will trigger other provinces to follow suit and, instead of using the B105 Code as a reference document, they will list it as one of their approved regulations.

The next obvious step will be to form a Deviation Committee, something that has been in great demand but impossible whilst we did not have an authority having jurisdiction. This Committee will be able to review requests for deviations to the requirements of the C.G.A.-B105-M87 Code. The deviations that are allowed will become proposed revisions which will be reviewed at the next B105 Code Committee meeting.

It is important to use this opportunity to tell you that no standard is "cast in stone", never to be revised or added to.

To complement the Installation Code, the Ontario Government's M.O.E. has compiled a set of guidelines which can be used to assist in the design, installation, inspection and training of plant operators.

Many questions have been presented to the B105 Secretary and the author, as Chairman, which relate directly to a key question, "How far do we have to go?" One question I'm sure you can relate to is, "What about the approval status of the digester gas controls and components?"

We have the B105 Installation Code, but we have no standards to regulate the construction of most of these controls and components, no standards which will allow for Canadian Certification of Flame Arrestors, Vacuum/Pressure Relief Valves, Sediment Traps, Drip Traps, and Flare Stack Burners, to name just a few.

C.G.A. has tested and certified automatic safety shut-off valves.

The processing of new standards is costly and time consuming but if that is the way we need to go, then there is criteria set up by the Standards Council of Canada that must be followed. To follow this criteria presents us with the problem of getting several manufacturers interested in working together towards the goal of Canadian Certification and Listing in the C.G.A. Directory of Appliances and Accessories.

Even though this criteria is set up for the processing of a new standard, the advent of modern technology has enabled C.G.A. to use interim measures for the Certification of Equipment for which there is no standard. We intend to use this method so as to continue making progress into a better future.

At present we have certain manufacturers who have a monopoly on the controls and components presently being used in the digester gas systems. We also have reports which indicate many improvements in the construction, performance and quality of the controls and components are urgently desired.

Recently C.G A. was requested by a digester gas equipment manufacturer to test and certify their equipment so that the C.G.A. certification symbol displayed on their product would allow it to be accepted.

Because there is no standard available C.G.A. will set up a task force to compile basic test criteria which will then be circulated to each provincial government involved for their review and comments. If the comments returned indicate a need, more requirements will be added to the basic test criteria. The basic test criteria will then be used by C.G.A. to test and certify the equipment submitted.

When this has been accomplished, the equipment which has been used and accepted by convention will be classed as non-certified. This means that for future projects only Canadian certified equipment will be accepted.

The next step to this is for it to be determined that a National Standard should be initiated for any one of the pieces of digester gas equipment.

The request for a new standard has to be submitted to the Interprovincial Gas Advisory Council [I.G.A.C.], who has to evaluate the request and determine whether or not the standard is needed. Once the I.G.A.C. gives C.G.A. the go ahead, it starts the process of the interested parties working together to produce the new standard.

Each new standard can be a "Key to a Better Future" and we owe ourselves and our neighbours that better future. By working together we can accomplish that goal.

IS OZONE IN YOUR FUTURE

Dr. Don Richard
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INTRODUCTION

Numerous studies have been performed in recent years investigating the effects of ozonation on the biodegradability of organic matter in treated wastewater effluent. It has been the general consensus of these investigations that ozonation tends to increase the biodegradability of refractory or non-biodegradable organic material (as measured in the short term). Various measurements of biodegradability have been proposed (1). The most common measure used has been the five-day carbonaceous biochemical oxygen demand ($CBOD_5$). Increases in $CBOD_5$ following ozonation have been believed to be due to increases in biodegradability. Few studies have investigated CBOD exertion as a function of time for unozonated and ozonated effluents. However, this is particularly important when studying treated wastewater effluents containing refractory organic matter because assumed CBOD relationships may not always hold true. The rate of CBOD exertion in treated effluent may be much slower than for comparable concentrations of original substrate and the assumed first-order relationship may be subject to deviations (2). Thus, a study of the effects of ozonation on CBOD as a function of time for an activated sludge effluent seems appropriate.

OBJECTIVE OF RESEARCH

The objective of this study was to investigate the effects of ozonation on the biodegradability of residual organic material in activated sludge effluent as measured by CBOD versus time and COD.

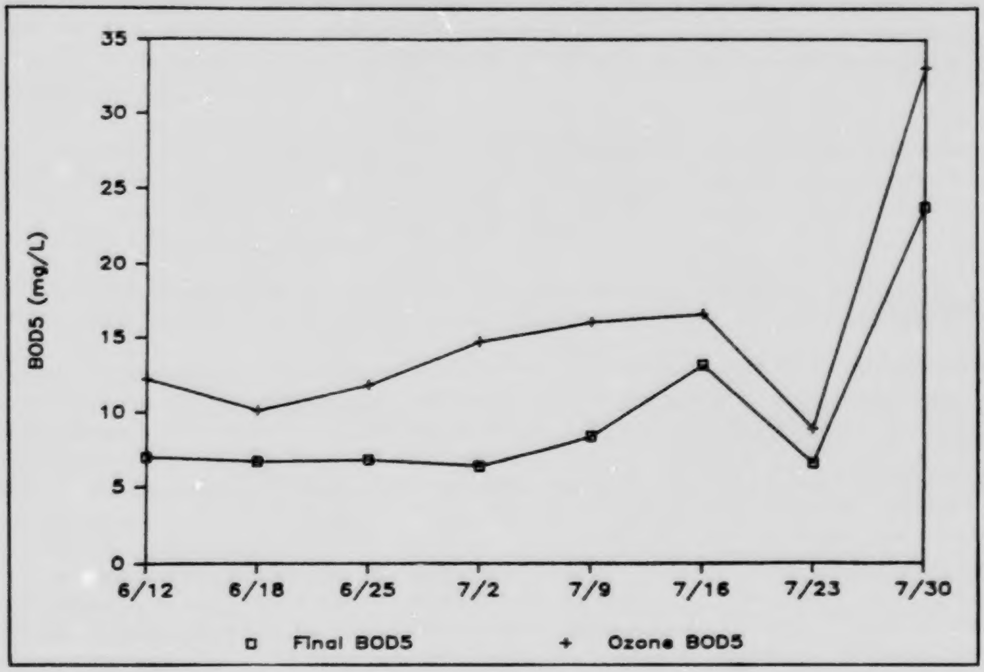
WASTE SOURCE

The City of Moorhead, Minnesota treats wastewater with a pure-oxygen activated sludge treatment plant. This treatment plant receives a wastewater flow of approximately 4 million gallons per day (mgd) from domestic and industrial sources. Domestic wastes from a population of about 32,500 contribute about 3 mgd. The major industrial contributor is a malt house located in Moorhead which contributes about 1 mgd. The malt waste treated is approximately 20% to 30% higher in chemical oxygen demand (COD), five-day biochemical oxygen demand (BOD_5), and total suspended solids (TSS) than typical municipal waste.

Treated effluent from the activated sludge process (final clarifier effluent) is currently disinfected with ozone prior to discharge to the Red River of the North. The BOD_5 measured on the ozone-disinfected effluent has been found to be consistently higher than the BOD_5 measured on the final clarifier effluent. Typical data from plant records for 1985 are shown in Figure 1.

FIGURE 1

Final Clarifier and Ozonated Effluent BOD$_5$ from Plant Data



METHODS AND MATERIALS

Sample Collection: Grab samples of pure-oxygen activated sludge secondary effluent were utilized for the study. All tests were performed within 12 hours of sample collection. All samples were filtered (Whatman 934-AH) prior to testing so that results are a function of the soluble organic compounds present.

Ozonation: Ozone was produced with a Welsbach T-186 laboratory ozone generator using a pure-oxygen gas source. The gas flowrate and voltage were adjusted to produce a feedgas ozone concentration of about 40 mg/L. A known quantity of the feedgas was applied to a two-liter sample in a sealed reaction chamber. Unreacted ozone was collected in a potassium iodide (KI) trap. Feedgas concentrations and quantities of unreacted ozone in the reactor offgas were determined using an iodometric method.

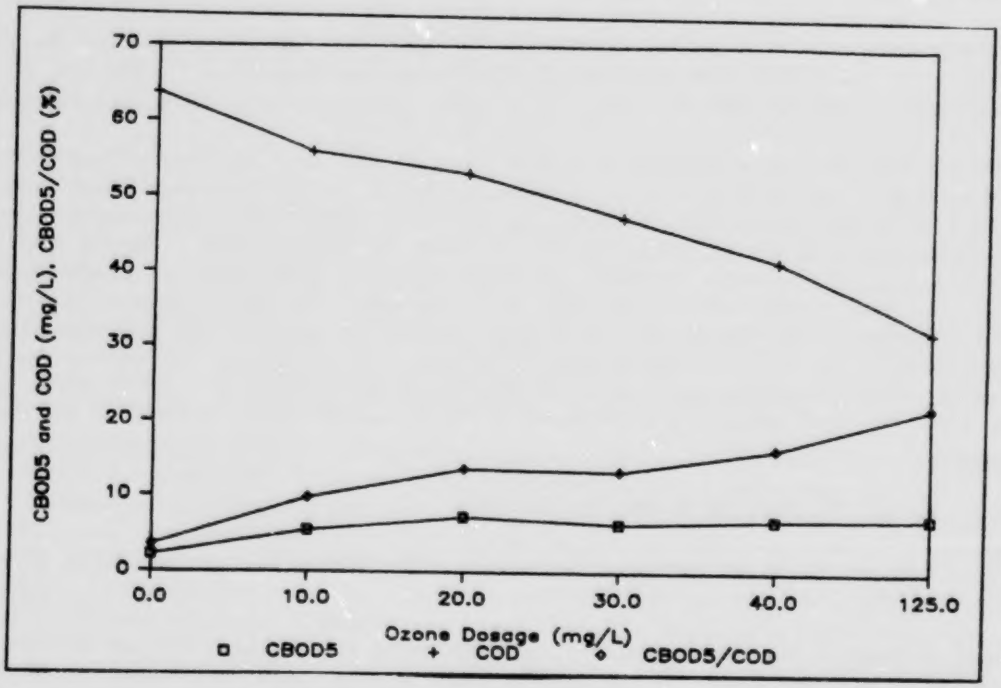
Biodegradability Characterization: Samples of secondary and ozonated effluent were subjected to oxygen demand tests to characterize biodegradability. CBOD$_5$ and COD tests were performed in accordance with Standard Methods, 15th ed., (1980). CBOD versus time relationships were determined using a bench-scale manometric biochemical oxygen demand apparatus. Results thus obtained were statistically analyzed and fit to first-order curves. All CBOD tests were performed at 20°C.

PHASE 1: $CBOD_5$ VERSUS OZONE DOSAGE

The Phase I objective was to investigate the relationship between $CBOD_5$ and ozone dosage. To accomplish this, ozone was applied at six dosages to samples of filtered secondary effluent. $CBOD_5$ and COD concentrations were measured on each sample. The results obtained are shown in Figure 2. It should be noted that some of the dosages applied were known to be excessive but the relationships for dosages higher than normal were of interest.

FIGURE 2

$CBOD_5$ and COD Versus Ozone Dosage



As observed in Figure 2, the $CBOD_5$ increase with ozonation is verified for this effluent. It is interesting to note, however, that at dosages higher than about 20 mg/L no further increase is observed. $CBOD_5$ increases initially and then achieves a constant, maximum value. COD decreases for all dosages applied. The $CBOD_5$/COD ratio increases for all dosages applied. All curves exhibit greater changes initially (at low ozone dosages) and successively smaller changes as ozone dosage is increased.

The data obtained in this phase provide two indicators of biodegradability. The first indicator is $CBOD_5$. Increases in $CBOD_5$ due to ozonation have been proposed to be due to increases in the biodegradability of residual organic matter present in the secondary effluent. Using this as a measure, increased biodegradability with ozonation is implied for this effluent. However, to use $CBOD_5$ as the only indicator of biodegradability certain assumptions must be satisfied. These assumptions are:

1. the rates of CBOD exertion (k) for both unozonated and ozonated effluents are equal, and

2. there are no deviations from the assumed first-order relationship for CBOD versus time for either effluent.

The significance of the first assumption is that for a given value of k, the $CBOD_5$ measured for any waste is a certain percentage of the ultimate carbonaceous biochemical oxygen demand ($CBOD_u$). When the CBOD rate constants are equal for unozonated and ozonated effluents the measured $CBOD_5$ values will represent the same percentage of the respective $CBOD_u$ values. Therefore, if ozone changes the nature of refractory organic matter by conversion to biodegradable organic matter an increase in $CBOD_u$ will be represented by a corresponding increase in $CBOD_5$.

The second assumption is necessary so that it is known that equivalent measures are being compared. If deviations from the typical first-order relationship exist for one of the effluents but not for the other the comparison of $CBOD_5$ values will be erroneous and misleading.

A different indicator of biodegradability that might be used with the data collected is the $CBOD_5$/COD ratio. This ratio is a relative measure of the amount of biodegradable organic matter present (indirectly represented by $CBOD_5$) to the total amount of organic matter present (represented approximately by COD) measured in terms of oxygen demand. As the amount of biodegradable material present increases in proportion to the total amount of organic material present, increased biodegradability is suggested. However, the use of this ratio requires careful attention due to the fact that both $CBOD_5$ and COD are changing over the range of ozone dosages investigated. In addition, because $CBOD_5$ is one of the parameters involved, the same assumption stated previously must apply in order for this ratio to be a valid indicator of biodegradability. Therefore, the verification of these assumptions is essential to any further discussion.

PHASE 2: EFFECTS OF OZONATION ON CBOD VERSUS TIME

The Phase 2 objective was to investigate the nature of the CBOD versus time curves for secondary and ozonated effluents.

As shown in Figure 3, the CBOD versus time relationship for this filtered effluent deviates from the typical first-order curve. No appreciable oxygen utilization was measured during the first three days of incubation. After the third day, oxygen depletion began, following a typical first-order curve. Three possible explanations for this three-day lag are proposed.

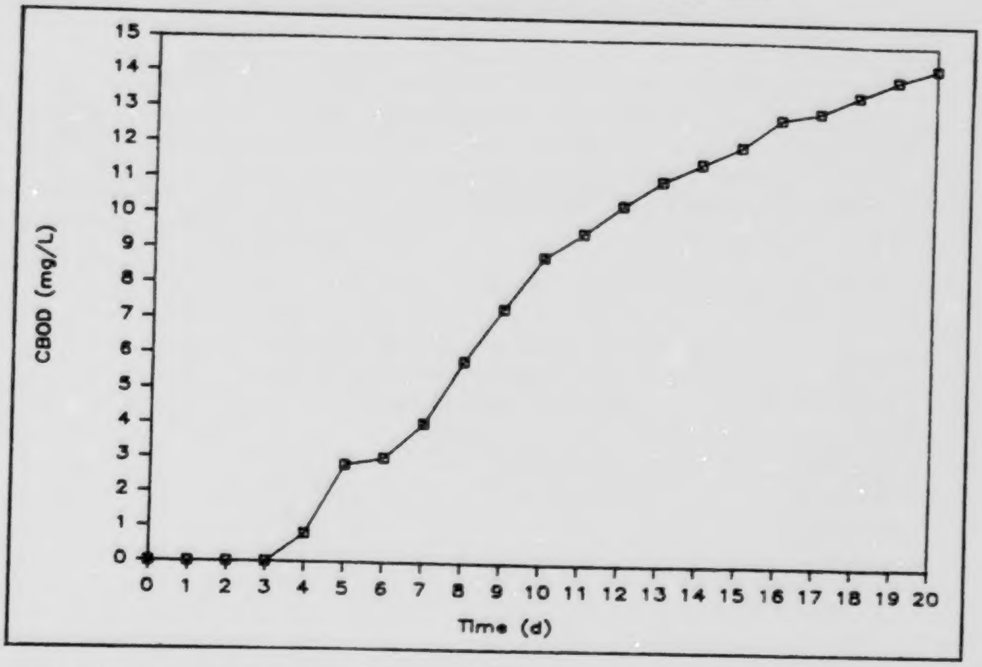
1. Sample filtering has removed a sufficiently large portion of the active microbial population so that normal oxygen uptake is impossible. After three days the microbial population has recovered to the point where normal uptake can once again occur.

2. The resident microbial population is initially unable to utilize the soluble, residual organic matter present. After three days the microorganisms have acclimated so that utilization of the substrate present is possible.

3. The presence of a toxic or inhibitory compound has resulted in the inability of the resident microbial population to function properly. After three days the microorganisms have adapted (acclimated) so that they are able to function in the environment.

FIGURE 3

CBOD Versus Time for Filtered Secondary Effluent



An investigation was undertaken to determine whether or not sample filtering may have removed a sufficiently large portion of the microbial population so as to cause the observed three-day lag. A sample of secondary effluent was collected and filtered. CBOD versus time relationships for five days were determined according to the method of CBOD determination presented in Standard Methods, 15th ed. (1980). Five dilutions were set with the filtered secondary effluent. Five additional, identical dilutions were also set but with the addition of primary clarifier effluent as seed. On each of the days 1 through 5 oxygen uptake (and hence, CBOD) was measured on one unseeded and one seeded dilution (correction for seed material taken into account). The results obtained are shown in Table 1 and Figure 4.

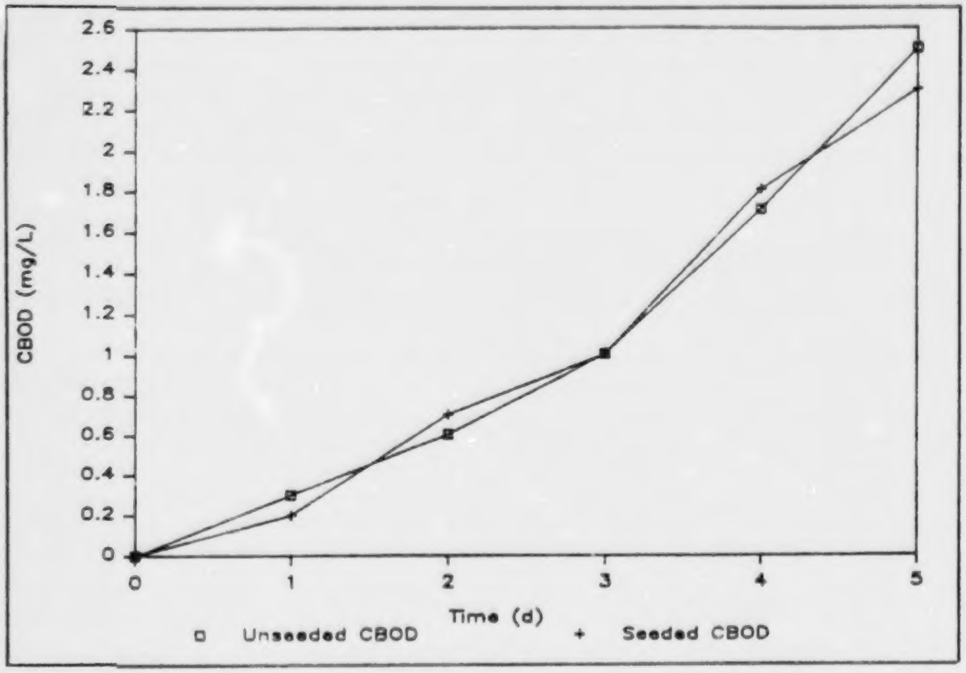
TABLE 1

CBOD Versus Time for Unseeded and Seeded Filtered Secondary Effluent

Day	Unseeded CBOD (mg/L)	Seeded CBOD (mg/L)
0	0.0	0.0
1	0.3	0.2
2	0.8	0.7
3	1.0	1.0
4	1.7	1.8
5	2.5	2.3

FIGURE 4

CBOD Versus Time for Unseeded and
Seeded Filtered Secondary Effluent



The addition of a supplemental microbial population (the primary effluent seed) did not change the observed oxygen uptake. Therefore, a sufficient microbial population must still remain after filtration and the first proposed explanation is proven invalid. It is also of interest to note that the amount of CBOD exerted at three days is very small (1.0 mg/L) and that the slope of the curve changes after the third day. The low CBOD exertion up to three days and the observed change in oxygen uptake after the third day agree with the data collected and previously shown in Figure 3.

With the first proposed explanation eliminated it could be concluded that either substrate problems exist or an inhibitory compound is present. Because the lag phase must be environmentally related (to substrate or chemicals present) and because normal oxygen uptake does begin after three days it is probable that a culture of microorganisms can be developed through acclimation that would be able to exhibit normal oxygen uptake beginning immediately. With this objective an acclimated seed was developed.

To develop the acclimated seed a sample of secondary effluent was collected and filtered. The sample was buffered to approximately pH 7 and gently aerated for five days. Aeration for five days ensured passing the lag phase and entering the phase of normal oxygen uptake. The seed thus developed was used in subsequent investigation.

The final objective of the Phase 2 investigation was to examine the effects of ozonation on the CBOD versus time relationship for filtered secondary effluent. The results obtained for an unozonated sample could be compared with those obtained for an ozonated sample and used to draw conclusions as to changes in biodegradability. The effects of utilizing the acclimated seed were also investigated.

A sample of secondary effluent was collected, filtered, and split into three portions. The first portion received no further treatment. The second portion was seeded with the developed acclimated microbial population. The third portion received an ozone dosage of 20 mg/L. After ozonation, the third portion was seeded with primary effluent. CBOD versus time relationships were determined for each sample portion. Corrections for seed were measured and applied to obtain the final results shown in Table 2.

TABLE 2

CBOD Versus Time for Secondary Effluent, Secondary Effluent With Acclimated Seed, and Ozonated Effluent

Day	Secondary Effluent CBOD (mg/L)	Effluent Acclimated Seed CBOD (mg/L)	Ozonated Effluent CBOD (mg/L)
0	0.0	0.0	0.0
1	0.0	1.3	1.5
2	0.0	2.1	3.3
3	0.0	3.4	4.3
4	1.3	4.7	4.9
5	2.0	5.4	6.3
6	4.0	7.2	7 7
7	5.0	8.1	8.5
8	6.3	8.9	8.7
9	7.5	9.9	9.7
10	8.3	10.6	10.2
11	9.0	11.1	10.7
12	10.0	11.6	11.1
13	11.0	12.1	11.8
14	11.3	12.1	12.1
15	12.3	12 6	12.3
16	13.0	13 1	12.6
17	13.3	13.3	12.6
18	13.3	13.3	12.6
19	13.8	13.8	12.6
20	14.0	14.1	12.6

The data obtained for all three relationships were fit to first-order curves using linear regression techniques. The linear regression model is as follows.

$$\ln(CBOD_r/CBOD_u) = -kt$$

where

$CBOD_r$ = CBOD remaining at time, t (mg/L)

$CBOD_u$ = ultimate CBOD (mg/L)

k = rate of CBOD exertion (d^{-1})

t = time (d)

The secondary effluent curve was fit to a first-order curve beginning with day 3. The equations thus obtained are given as (1) for the unozonated secondary effluent, (2) for the secondary effluent with acclimated seed, and (3) for the ozonated secondary effluent.

$$CBOD_t = 19.11(1 - e^{-.082(t-3)}) \qquad (1)$$

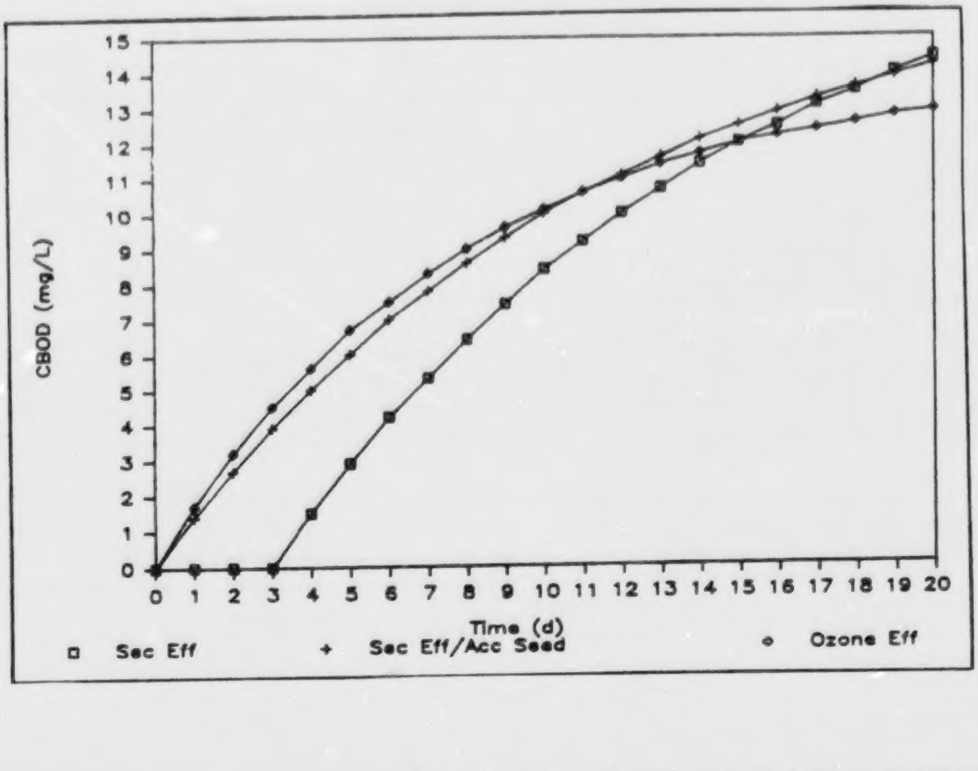
$$CBOD_t = 17.29(1 - e^{-.086t}) \qquad (2)$$

$$CBOD_t = 13.99(1 - e^{-.128t}) \qquad (3)$$

These curves are plotted in Figure 5.

FIGURE 5

CBOD Versus Time for Secondary Effluent, Secondary Effluent With Acclimated Seed, And Ozonated Effluent



SAS, a statistical analysis computer software package was used to analyze how well the data fit these curves. Pertinent results of this analysis are shown in Table 3. The significance level (alpha) or probability for the first-order relationship being incorrect is 0.0001 or 0.01% as determined by the F statistic. The corresponding F values are given in Table 3. The values obtained for R^2, the coefficient of determination (the square of the correlation coefficient, 4), are all near the maximum value of 1 for a perfect mathematical fit.

TABLE 3

Statistical Data For the CBOD Versus Time Curves

Effluent	BOD_u (mg/L)	k (d^{-1})	F	Alpha	R^2
Unozonated	19.11	0.082	1815	0.0001	0.9918
Acclimated Seed	17.29	0.086	1553	0.0001	0.9885
Ozonated	13.99	0.129	645	0.0001	0.9728

The three-day lag was again observed for the unozonated secondary effluent. The net result of the addition of the acclimated seed was to shift the origin of the CBOD versus time curve from day 3 to day 0. All other parameters describing the secondary effluent and secondary effluent with acclimated seed curves are nearly identical. The ultimate CBOD values agree to within 10% and the rates of CBOD exertion, k, agree to within 5%. Therefore, within acceptable limits of error, it can be concluded that these are, in fact, the same curves with the lag eliminated by the acclimated seed. The elimination of the lag with the acclimated seed does not indicate exactly what has caused it. However, it verifies that the resident microorganisms can adapt to the conditions present. It also shows that the CBOD versus time relationship for filtered secondary effluent beginning at day 0 is not representative of the true content of biodegradable matter. Thus, the curve obtained for the secondary effluent with acclimated seed will be useful for comparison to the results obtained for the ozonated effluent.

Significant differences in the nature of the CBOD versus time relationship were observed for the ozonated effluent. The lag has been eliminated. Ozonation has resulted in a change in the environment. This change could be the result of either 1) modification of the nature of the substrate, or 2) elimination of any inhibitory conditions previously present. This change has resulted in conditions favorable to the resident microbial population for degradation of the organic matter remaining in the effluent. Thus, immediate oxygen uptake begins at day 0.

The elimination of the lag is accompanied by changes in both $CBOD_u$ and k. Using the secondary effluent with acclimated seed relationship for comparison, with the ozonation process (with a dosage of 20 mg/L) has reduced the $CBOD_u$ by nearly 20% and increased the rate of CBOD exertion by 50%. The increased rate of oxygen utilization most probably signifies a change in the structure of the original organic substrate(s). Such a change could be the result of oxidation by ozone converting molecules with structures difficult or impossible to degrade to molecules readily degradable. This allows the resident microbial population to utilize these molecules without adaptation (acclimation). Thus, based on the results obtained, it is highly probable that the net effect of ozonation is to increase the biodegradability of the organic matter present in this activated sludge effluent.

The results obtained from this investigation must be viewed with respect to the relationships obtained in both phases of this investigation. In Phase 1, $CBOD_5$ was observed to initially increase with increased ozone dosage. In Phase 2, it has been determined that this increase is primarily due to a shift in the CBOD versus time curve and not due to an increase in $CBOD_u$ as was previously believed. After this shift has occurred, no further appreciable increase in $CBOD_5$ takes place; a maximum value is achieved and maintained. This indicates that no matter how high the ozone dosage is, after the initial $CBOD_5$ increase due to the curve shift no further significant increases will take place. Thus, the inadequacy of the $CBOD_5$ test (a one-day measurement) as an indicator of biodegradability is noted.

It is evident that ozone affects both the non-biodegradable and biodegradable organic fractions. In Figure 8, the total length of the bar represents the COD concentration. At the corresponding heights, the concentrations of measured $CBOD_5$ and $CBOD_u$ are shown. The difference between the COD and $CBOD_u$ heights represents the non-biodegradable organic fraction in terms of oxygen demand. The $CBOD_u$ represents the biodegradable organic fraction in terms of oxygen demand.

FIGURE 8

Oxygen demand Characteristics for Secondary Effluent,
Secondary Effluent with Acclimated Seed,
and Ozonated Effluent

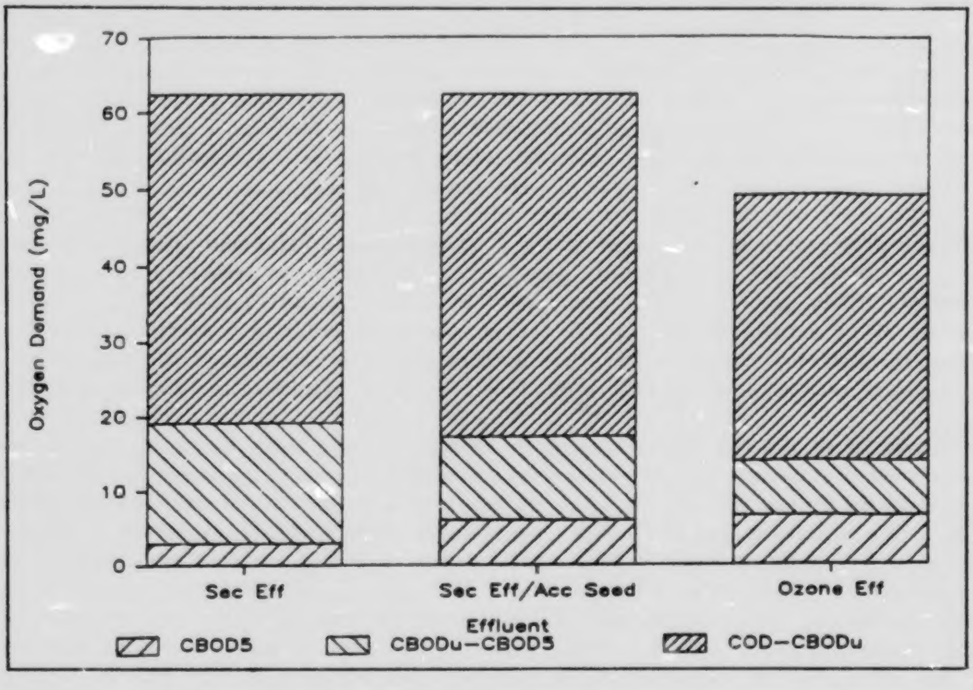


TABLE 4

Oxygen Demand Characteristic for Secondary Effluent, Secondary Effluent with Acclimated Seed, and Ozonated Effluent

Sample	$CBOD_5$ (mg/L)	$CBOD_u$ (mg/L)	COD (mg/L)	$(COD-CBOD_u)$ (mg/L)
Unozonated	2.9	19.1	62.3	43.2
Acclimated Seed	6.0	17.3	62.3	45.0
Ozonated	6.6	14.1	49.1	35.1

The comparison of $CBOD_5$ values is not a comparison of equivalent representative measures of biodegradability. Rather it is a comparison of secondary effluent $CBOD_2$ to ozonated effluent $CBOD_5$. When the lag is eliminated or compensated for (as with acclimated seed) the actual change in $CBOD_5$ with ozonation is negligible. Thus, not understanding the effects of ozonation on the CBOD versus time relationship leads to erroneous reasoning as to changes in biodegradability. Yet as previously stated increased biodegradability is suggested by the increased rate of CBOD exertion, k, a better indicator of biodegradability.

CONCLUSIONS

The following conclusions are the result of this investigation. These conclusions apply specifically to the effluent investigated and ozone dosages applied and are not necessarily applicable to all types of effluent for all possible ozone dosages.

1. Activated sludge secondary effluent from the Moorhead, Minnesota Wastewater Treatment Facility is characterized by the presence of soluble, residual organic matter, a portion of which is difficult to degrade and the remainder of which is essentially non-biodegradable. This residual organic matter is a function of the original waste treated as well as the type of treatment employed.

2. The presence of difficult to degrade organic material results in a three-day lag in the CBOD versus time relationship for this effluent.

3. It is possible to develop an acclimated microbial population from the resident population which is capable of degrading under normal conditions at least a portion of this soluble, residual organic matter. This suggests that different treatment modes may affect removal efficiencies. However, a portion of the residual organic matter will undoubtedly remain as non-biodegradable in character.

4. It is the presence of the difficult to degrade organic matter that has produced the increases in $CBOD_5$ following ozonation. The increases observed for this effluent are the result of a shift in the CBOD versus time relationship following ozonation and not due to an increase in $CBOD_u$.

5. Ozonation significantly affects the nature of the residual organic matter. Increased biodegradability is suggested by the increased rate of oxygen uptake (CBOD exertion) following ozonation.

6. Ozone was found to affect both non-biodegradable and biodegradable organic matter. Decreases in both non-biodegradable and biodegradable oxygen demand indicate improved effluent quality following ozonation.

7. The $CBOD_5$ test as an indicator of changes in biodegradability for unozonated and ozonated effluents is inadequate. Here, this resulted in the comparison of secondary effluent $CBOD_2$ to ozonated effluent $CBOD_5$; two entirely different measures.

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WATER POLLUTION CONTROL IN ONTARIO

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PROVINCIAL WATER QUALITY OBJECTIVES

In Ontario, wastewater treatment requirements are governed by the "Water Management Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment". The water management goal is "to ensure that the surface waters of the province are of a quality which is satisfactory for aquatic life and recreation". Although minimum treatment requirements may apply in principle, effluent requirements are established on a case-by-case basis in reference to the characteristics of the receiving water body and its intended uses. The surface waters of Ontario are put to many uses and each use has specific water quality requirements. Water quality must be managed, preserved and restored where necessary to permit the greatest number of uses, based on the best interests of the people of Ontario. In addition, since Ontario borders on inter-provincial and international waters, the Province's programs must consider the specific Water Quality Objectives of the Great Lakes Water Quality Agreement between Canada and the United States. Also, under the Canada-Ontario accord, Ontario must establish and enforce effluent requirements for specific industrial groups and pollutants.

Provincial Water Quality Objectives have been established for a number of parameters such that water which meets set quality criteria is satisfactory for aquatic life and recreation. This quality is suitable for most other beneficial uses, such as drinking water supply, and agriculture. If better water quality is required to protect other beneficial uses in a given location, appropriate criteria are determined and applied to that location.

Surface water quality management in Ontario is directed by two primary policies:

1) in areas which have water quality better than the Provincial Water Quality Objectives, water quality shall be maintained at or above the Objectives; and,

2) water quality which presently does not meet the Provincial Water Quality Objectives shall not be degraded further and all practical measures shall be taken to upgrade the water quality to the objectives.

The Provincial Water Quality Objectives are a set of narrative and numerical criteria designed for the protection of aquatic life and recreation in and on the water. They represent a desirable level of water quality that the Ministry strives to maintain in surface waters of the Province. They are often the starting point in deriving wastewater effluent requirements.

EFFLUENT REQUIREMENTS

For a desired level of water quality, every river or lake has a definable dilution, dispersion or self-purification capacity for receiving waste discharges. Efficient use of this capacity is a key to optimizing water pollution control programs. The emphasis of the Ministry's Water Quality Management Program is to set effluent requirements based on the waste assimilation capacity of the receiving water and the Provincial Water Quality Objectives, with consideration also given to federal and provincial effluent regulations or guidelines and controls on non-point sources of pollution.

A waterbody's capacity for receiving wastes depends upon the characteristics of the waste and a number of physical, chemical and biological factors, such as flow or volume of the receiving water and waste discharge, depth and width of receiver, algae growths, benthic deposits, reaeration capacity, etc.

Receiving water assessment techniques are used to estimate its assimilative capacity and from there, to establish effluent requirements such that Provincial Water Quality Objectives are met. Depending upon the circumstances, either simple dilution formulae or more complex mathematical models may be used. Pertinent manuals for conducting stream surveys and for establishing assimilative capacity using the modelling approach have been published by the Ministry of the Environment.

In establishing effluent requirements then, appropriate site-specific receiving water assessments are conducted to determine the required effluent quality based on the waste assimilation capacity of the receiver. This requirement will then be compared to federal effluent regulations and provincial effluent regulations or guidelines. The more stringent requirements will apply.

For proposed new or expanded facilities, the effluent requirement so established is incorporated into a Certificate of Approval for the facility discharge, which specifies both allowable concentration and loading of individual effluent parameters.

For existing discharges in areas with water quality worse than the Provincial Water Quality Objectives, a pollution control program is developed for each discharger such that the required effluent quality will be met on an approved time schedule.

Effluent requirements based on the above procedures may be established for municipal and industrial point and urban and rural non-point sources.

MUNICIPAL WASTEWATER TREATMENT PROGRAMS

A total of 403 sewage treatment facilities, with a combined approximate capacity of 5.9 million cubic metres per day serve some 1.7 million persons or about 83% of the total population of Ontario. The Ministry of the Environment operates 232 of these sewage works with the remainder being operated by municipalities.

All sewage treatment plants in the province provide at least primary treatment while 85% provide secondary treatment. Ninety-one percent (91%) of the province's total sewage treatment capacity receives phosphorus removal to a level of at least 1 mg/L.

For municipal sewage treatment works, effluent requirements are established on a case-by-case basis considering the characteristics of the receiving water and its intended uses. However, Ministry of the Environment (MOE) Policy 08-01 which addresses the Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters establishes "secondary" treatment as the normal level of treatment required. Relaxation of the normal level of treatment will be allowed on a case-by-case basis only when it can be demonstrated through a receiving water assessment study that the provincial Water Quality Objectives will be met and no water uses will be impaired. Under no circumstances will the level of treatment required be less than primary.

Higher levels of treatment than secondary, up to and including no discharge to surface waters may be required as established through appropriate site-specific receiving water assessments.

All effluent requirements are incorporated into Certificates of Approval.

INDUSTRIAL WASTEWATER CONTROL PROGRAMS

There are some 12,000 industrial discharge points throughout the Province of Ontario of which some 2,000 are major non-sewer or direct to receiving water discharges.

In Ontario, control of water pollution from direct industrial dischargers is achieved through the Canada-Ontario Accord for Environmental Protection. Under the Accord, Ontario has agreed to adopt pollution control requirements developed for specific industrial sectors. Where national effluent requirements do not meet Ontario's objectives for water quality based upon receiving water assessment studies, more stringent requirements are imposed.

Effluent requirements are applied at start-up for all new industrial installations or installations undergoing process modification or expansion. In all other cases national effluent requirements, for those industries where national guidelines or regulations exist are applied as a minimum as rapidly as possible to meet agreed objectives and time schedules. Permissible effluent levels are included in a Certificate of Approval for each discharge.

For existing plants where a pollution problem with significant environmental impact has been identified the Ministry will initiate a Pollution Abatement Program with the industry. Under this program, the industry is required to prepare a report outlining the technical options available for correcting the problem. In preparing this report, the company addresses such factors as technical capability, required capital investment, operating costs, economic feasibility, socio-economic implications and environmental benefits. Once this report has been prepared, specific abatement objectives and schedules are jointly developed and agreed upon.

Compliance may then be approved under a voluntary plan, a formal Program Approval, a Requirement and Direction or a Control Order. Failure to comply with program requirements can lead to prosecution under the Environmental Protection Act and the Ontario Water Resources Act.

Industrial discharges to municipal sewers are controlled under the Industrial Sewer Use Control Program. Sewer use control is the responsibility of individual municipalities such that industrial discharges will not adversely affect the sewer system, the treatment process or the quality of effluent. Most municipalities have developed sewer use by-laws regulating the excessive discharge of certain harmful materials to sewer systems. These by-laws vary among municipalities but are based upon the Model Sewer Use By-law developed in 1975 jointly by the MOE and the Municipal Engineer's Association of Ontario.

An Interagency Industrial Sewer Use Control Review Committee was established in 1985 to review the adequacy of the existing by-law program and identify means of controlling hazardous organic and inorganic contaminants. Models are being developed which will permit municipalities to allocate loadings of individual contaminants to the sewer system such that effluent and sludge quality requirements may be met. This Committee developed a revised interim model sewer Use By-law in 1988 which revised limits of some of the existing and expanded the number of parameters.

Ontario's approach to industrial pretreatment at this time is not to impose uniform pretreatment standards, but instead, to take into account the ability of the municipal sewage treatment plant to treat the industrial contaminants. However, significant modifications to Ontario's programs of pollution abatement from industrial and municipal sources are currently underway. These are addressed in the following sections.

MUNICIPAL-INDUSTRIAL STRATEGY FOR ABATEMENT (MISA)

Ontario's initiatives have demonstrated that environmental improvement and municipal and industrial growth can be compatible. Significant reductions in waste discharges have occurred over the past two decades in spite of rapid urbanization and overall production increases by industry and recurring recessionary factors in the economy.

Past pollution abatement programs however, have been directed primarily at conventional parameters such as Biochemical Oxygen Demand, Suspended Solids, Phosphorus and Ammonia. In certain instances some heavy metals, and organic compounds were also considered but little attention was paid to so-called toxic and hazardous organic and inorganic contaminants.

In June of 1986, the Ontario Ministry of the Environment announced a new program - The Municipal-Industrial Strategy for Abatement (MISA) - with the goal of further cleaning up Ontario's waterways.

MISA is a two-stage process to address pollution control of both conventional pollutants and hazardous contaminants from industrial and municipal point source discharges. Regulations will be used to limit the pollution loadings to surface waters. Initially effluent criteria will be established on the basis of best available technology in major industrial sectors and the municipal sector. Ultimately, effluent requirements will be based on the most restrictive of either technology-based or receiving water assessment-based requirements.

For each of the sectors, two regulations will be developed. Initially, discharges will be subject to a monitoring regulation which will require them to identify and measure concentrations and loadings of contaminants in their effluents. Subsequently an abatement regulation, based on the best available technology economically achievable will be developed on a sector-by-sector basis which will specify allowable concentrations as well as loadings of pollutant discharges for each discharger.

More stringent effluent limits may be established for specific sites to protect sensitive water bodies on the basis of site specific receiving water assessment studies.

Effluent limits for each discharger in each sector will be reviewed periodically to determine what further pollutant reductions are possible as a result of improved technology, or are required on the basis of receiving water impact of that particular discharge.

During the initial three-year implementation period of MISA it will address all major point sources of pollution to Ontario's surface waters. Ultimately the program will regulate all effluent wastewater discharges from industries and municipalities throughout Ontario.

The three year implementation period will address the municipal sector and all industries in the following sectors: petroleum refining, organic and inorganic chemical manufacturing, pulp and paper, metal mining and refining, industrial minerals, iron and steel manufacturing, and electric power generation.

Under MISA, the Ministry of the Environment is also reviewing various alternatives for a more effective and efficient sewer use control program. Such a program could retain the present strategy of municipal control or could place the control at the provincial level and establish pretreatment requirements for designated industrial sectors.

URBAN DRAINAGE MANAGEMENT PROGRAM

Urbanization increases both the volume and rate of runoff by converting pervious soil to impervious roads, parking lots and rooftops. Reduced infiltration into the soil results in lower groundwater tables and reduced base flows in streams. Accelerated runoff results in higher flood flow peaks and channel erosion. Lack of a comprehensive approach to address these problems in the past has led to inadequate attention to urban drainage by most municipalities.

The Urban Drainage Management Program now in its implementation stage is designed to complement the existing land use planning process in Ontario and will encourage municipalities to follow good drainage planning during development and apply good practices in managing stormwater. Initially the program will be introduced on a voluntary basis with an extensive training program to alert municipalities and developers of its benefits and techniques.

Good planning includes preparation of Watershed Urban Drainage Targets and constraints; preparation of Master Drainage Plans; and preparation of Stormwater Management Plans. Although the emphasis is on quantity control, stormwater quality and treatment alternatives are considered where necessary. Good drainage practices include major and minor drainage system design and erosion and sediment control during construction.

Watershed Urban Drainage Targets are developed as components of watershed plans which identify quantity constraints and quality control requirements to protect present and projected future use of the receiving waters.

Municipalities will be encouraged to develop Master Drainage Plans which will respect the constraints identified in the watershed plans. These plans will typically contain allowable runoff rates, stormwater flow routes, approximate size and location of detention ponds, etc.

Stormwater Management Plans will be prepared by proponents of development as part of the subdivision design and approval process. These plans will implement the requirements set forth in Master Drainage Plans and drainage design will be integrated with the land use layout. They will also indicate the impact of the proposed development on water quantity and quality and specify mitigative measures.

An urban drainage system is comprised of two complementary systems; the major and minor drainage system design. The minor system is operational during low intensity (frequent) storms, whereas, the major system becomes operational when the capacity of the minor system is exceeded during high intensity (rare) storms. When the major and minor systems are properly designed and integrated, they will provide a high level of protection against flooding and keep any inconvenience at a reasonable level.

Erosion and sedimentation caused by urban construction results in adverse impact on off-site drainage systems and watercourses, degradation of water quality and damage to aquatic life. Erosion and sediment control practices such as protection of exposed surfaces, reduction in slopes and gradients, use of impoundments, etc., are most effective if applied at the time of construction. Consequently, municipalities are encouraged to impose good erosion and sediment control practices on developers during the subdivision construction phase.

POLLUTION CONTROL PLANNING

While the Urban Drainage Management Program is directed at good planning and practices for managing stormwater in new developments, the Pollution Control Planning process has been established to address pollution problems from multiple sources within an existing municipality or even perhaps on a watershed basis involving several municipalities, industries and agricultural components.

The analysis of an observed water quality problem should indicate which parameters are in violation of specific water quality objectives. From this, a target for control of pollution sources can be established. When a municipality has multiple pollution sources, or when industrial and/or rural sources interact, the problem becomes increasingly complicated with respect to which sources should be controlled and to what degree.

The Pollution Control Planning process assists in the systematic analysis of control options of multiple pollution sources in order to arrive at the most cost-effective solution to meet the specified water quality objectives.

Under the process, the relative loading of problem pollutants from each source is assessed and compared under wet and dry weather conditions. Various control options can then be assessed with respect to their effectiveness in pollutant load reduction and the corresponding water quality improvement. Pollution models such as STORM or SWMM and assessment models such as QUAL-II or RECEIV may be used. Factors related to economic, environmental and social constraints are taken into account. The cost-effectiveness of each of the various options is then determined and the optimum combination is selected. Frequently, the optimum option may include partial control of several sources. The traditional approach of upgrading the sewage works may provide a solution to the water quality problem but at a much higher cost than such possible controls as combined sewer overflow detention, upgraded industrial treatment and/or stormwater treatment.

AGRICULTURAL LAND USE POLLUTION CONTROL PROGRAMS

Water quality concerns from agricultural practices arise from a number of diffuse sources of pollution. Sheet and rill erosion from cropland as well as bank erosion caused by cattle access to streams or surface runoff, may cause unacceptably high levels of suspended sediment in streams. Such sediment frequently carries with it high concentrations of pesticides and phosphorus. High levels of phosphorus and nitrogen may also reach receiving streams in runoff from manured fields and unprotected manure storage areas. Inadvertent or illegal connections to field drainage tile may also carry milk-house waste and barnyard manure directly to streams or municipal drains.

In Ontario, concurrent federal and provincial jurisdiction exists to legislate in relation to agricultural pollution control, although undoubtedly, the Ontario Water Resources Act has the greatest powers over the control of water pollution from agricultural operations. This Act prohibits the discharge of any material to, or on land adjacent to, a watercourse in such a manner that water quality impairment may result. The Act clearly applies to the discharge of wastes from confined livestock operations, the application of fertilizers and manures and their subsequent runoff, and the use of pesticides.

Recent detailed studies of several watersheds in the province have determined the adverse impact of agricultural operations on some receiving waters. Programs are being established to reduce these impacts.

Such programs include Enhanced OSCEPAP (Ontario Soil Conservation and Environmental Protection Assistance Plan) under which special grants are provided to farmers for livestock waste management and soil erosion control projects in designated environmentally sensitive areas. The Phosphorus Load Reduction Plan for Lake Erie will evaluate the effectiveness of specific remedial practices in controlling phosphorus loads from agricultural soils. In addition, demonstration sites have been or are being developed in several watersheds to illustrate the benefits of conservation practices in agriculture.

SUMMARY

In summary, Ontario's programs of water pollution control extend to all sources including industrial and municipal point sources and municipal and rural non-point sources.

Although water management in Ontario is governed by the Provincial Water Quality Objectives, discharges are assessed on a case-by-case basis and appropriate effluent requirements take into account the characteristics and beneficial uses of the receiving water.

Ontario's new MISA program, through the development and imposition of sector specific monitoring and compliance regulations, will provide a much more formal and rigorous approach to water pollution control in Ontario. The basic principle of establishing specific effluent requirement for each receiving body of water will however continue to apply such that protection of its beneficial uses is assured.

SHELL PEACE RIVER
SOURCE WATER TREATMENT FACILITY

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INTRODUCTION

The purpose of this paper is to discuss the design of the Shell Canada Limited Peace River Expansion Project (PREP) I source water treatment facility located near the town of Peace River, Alberta, Canada. Particular attention was given during the design of the facility to the exceptionally high concentrations of suspended solids that occur in the Peace River.

The Shell Canada Limited PREP I project is an expansion of the existing Peace River In Situ Pilot Project PRISP from an existing average production rate of $200 m^3/d$ to $1600 m^3/d$ of bitumen. The PREP I expansion is located adjacent to and fully integrated with the PRISP project. The central plant is located approximately 55 km northwest of the town of Peace River. (See Figures 1 & 2)

FIGURE 1

Flow of the Peace River Through B.C. and Alberta

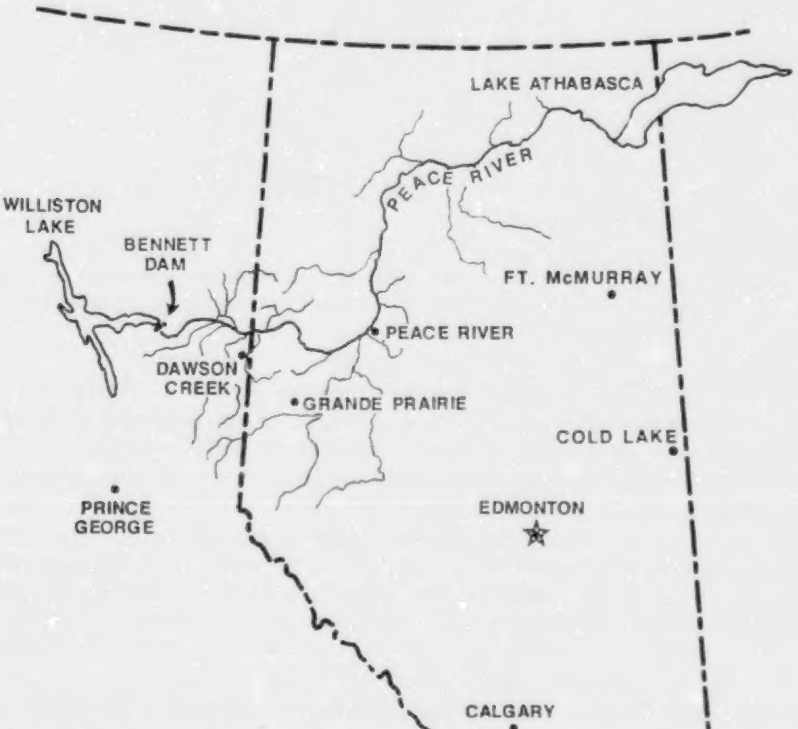
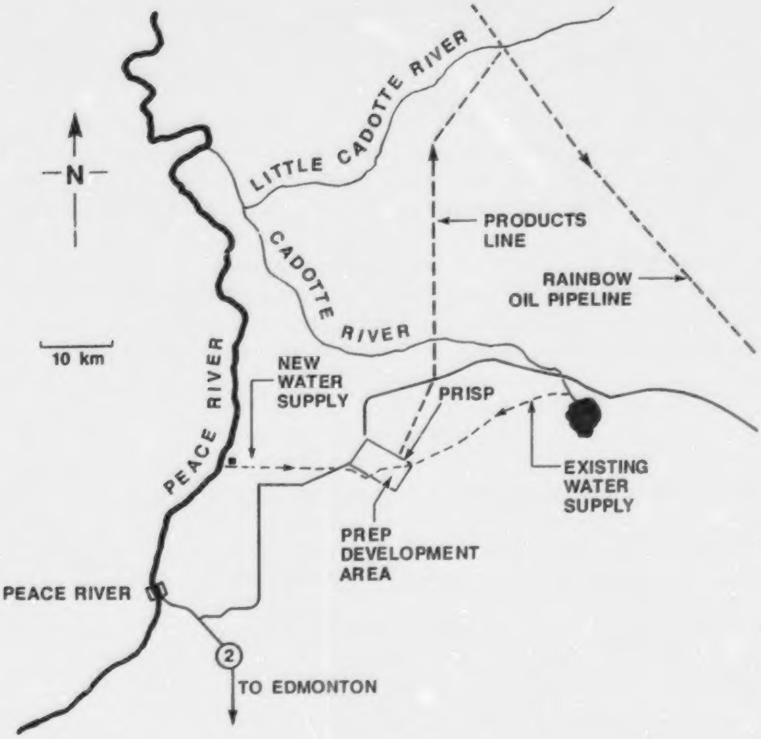

LAKE ATHABASCA
PEACE RIVER
WILLISTON LAKE
BENNETT DAM
FT. McMURRAY
PEACE RIVER
DAWSON CREEK
GRANDE PRAIRIE
COLD LAKE
PRINCE GEORGE
EDMONTON
CALGARY
BRITISH COLUMBIA
ALBERTA

FIGURE 2

Location Map Shell PREP


N
LITTLE CADOTTE RIVER
CADOTTE RIVER
PRODUCTS LINE
RAINBOW OIL PIPELINE
10 km
PEACE RIVER
NEW WATER SUPPLY
PRISP
EXISTING WATER SUPPLY
PREP DEVELOPMENT AREA
PEACE RIVER
2
TO EDMONTON

The source water treatment facility for PREP is located on the bank of the Peace River approximately 25 km west of the Central Plant. Treated water is supplied to the Central Plant via a pipeline.

The treatment facility at the river removes floating debris on travelling screens, while returning any entrained fish back to the river. The high concentrations of suspended solids are removed by clarification and filtration, producing water of potable quality. The filtered water is deaerated and stored in a tank with a floating roof prior to being pumped to the central plant via the pipeline.

The source water plant was designed to facilitate two future expansions from the initial capacity of 7,000 m^3/day (1,283 USGPM) to the ultimate capacity of 30,000 m^3/d (5,503 USGPM).

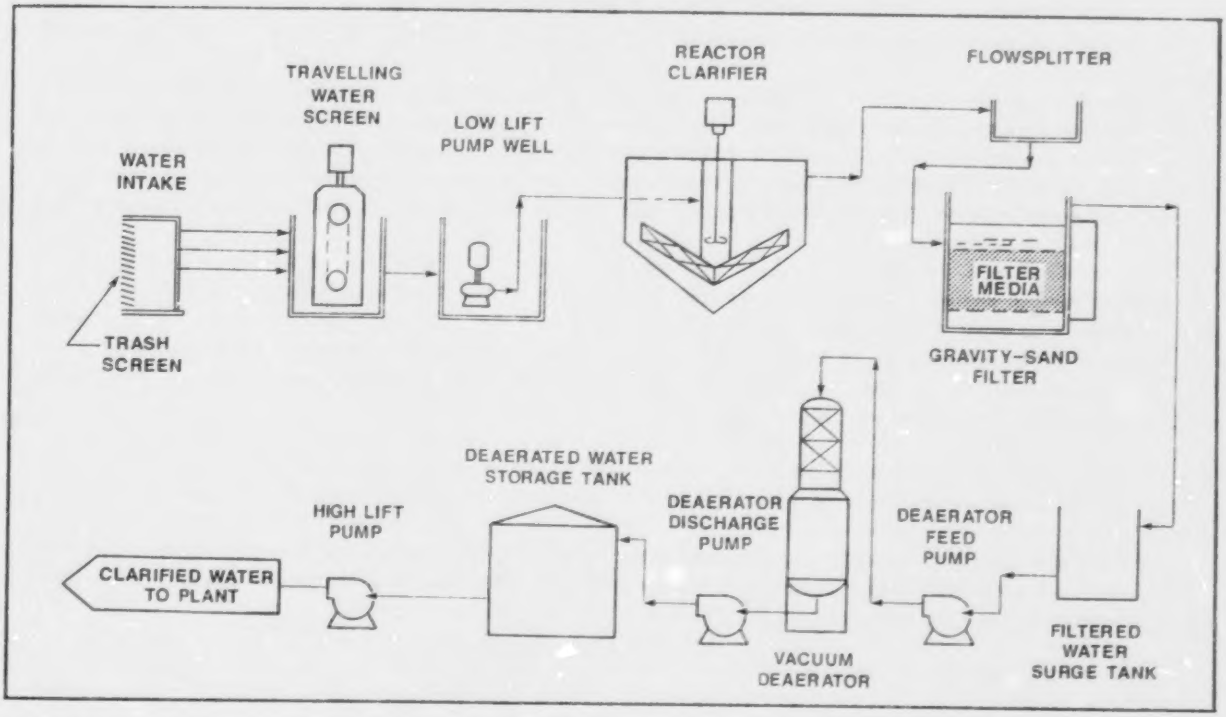
GENERAL

The feedwater for PREP Stage I high pressure steam generators is a blend of softened de-oiled produced water and softened fresh water. The two sources of fresh water come from the existing Cadotte Lake system and the new Peace River system. This presentation relates to the new Peace River source water supply facility.

As shown in the process schematic (Figure 3), the Peace River water flows through a trash rack into a concrete intake structure and then to the low lift pump house via four 16" diameter pipes. After removal of floating debris and the return of fish to the river, the raw water is pumped by two 100 percent low lift pumps to the clarifier for treatment. Chlorine, alum and coagulant aids are injected into the clarifier influent to enhance the processes of coagulation and floculation of suspended solids. The clarifier effluent overflows to the filter splitter box which equally splits the flow among four gravity filters which operate in parallel. The gravity filters remove the pin floc carryover from the clarifier effluent before discharging water to the filtered water surge tank.

FIGURE 3

Flow Schematic
Source Water Supply and Treating


TRAVELLING WATER SCREEN
REACTOR CLARIFIER
FLOWSPLITTER
LOW LIFT PUMP WELL
WATER INTAKE
FILTER MEDIA
TRASH SCREEN
GRAVITY-SAND FILTER
DEAERATED WATER STORAGE TANK
DEAERATOR DISCHARGE PUMP
DEAERATOR FEED PUMP
HIGH LIFT PUMP
CLARIFIED WATER TO PLANT
VACUUM DEAERATOR
FILTERED WATER SURGE TANK

The filtered water is pumped to the two stage vacuum deaerator tower for removal of oxygen and other non-condensable gases before delivery to the deaerated water storage tank by means of the deaerator discharge pump. A common spare pump is provided as backup for both the deaerator feed pump and discharge pump.

Two 100 percent high lift pumps are used to deliver the pretreated water to the Central Plant via a 25km pipeline.

The flow of raw water to the clarifier is controlled by a cascade flow control loop with the water level in the deaerated water storage tank generating the set point on the flow controller.

The clarifier inlet flow control system establishes both the maximum and minimum flows while controlling the rate of the flow change in order to eliminate surges to the clarifier. The system was designed to allow the flow to be increased to 17800 m^3/d in Phase II by the addition of a low lift pump, settling tubes in the clarifier and a high lift pump.

The ultimate plant output capacity of 30,000 m^3/d requires expansion of the treatment building to accommodate additional treatment facility equipment including three low lift pumps, one clarifier, one splitter box, four filters, filtered water surge tank, three deaerator feed and discharge pumps, deaerator, and three high lift pumps.

RAW WATER INTAKE SYSTEM

The intake system is sized for a design flow of 35,000 m^3/d with an approach velocity of approximately 0.4 m/s. The submerged intake structure was designed to withstand ice flow and prevent freezing. This structure consists of a trash rack 3400 mm wide by 1870 mm high and containing 40 mm square galvanized bars.

The pump house is comprised of three concrete compartments - the entrance chamber, screening chamber and the low lift pump well. All three compartments are interconnected and equipped with sluice gates to permit isolation. Water flows from the intake structure to the entrance chamber via four 16" diameter FRP pipes. Backflushing facilities are provided for each of the intake pipes and screens for cleaning purposes. The water for the backflushing operation is provided by the low lift pumps.

The screen chamber is divided into two chambers, one chamber has a travelling screen and the other a sliding frame screen for by-pass. Design of the sliding frame screen was such that it could be removed and replaced by a second travelling screen without modifications to the structure. The travelling screen is equipped with fish catching buckets and a soft spray feature to return any entrained fish back to the river.

Each travelling screen is 1880 mm wide by 11250 mm high and constructed of stainless steel material consisting of 2.032 mm diameter wire mesh with 10 mm square openings.

The pump bay is designed to accommodate the two low lift pumps used for the initial plant as well as four additional pumps for future phases. The chlorine diffusers prevent biological growth in the pump bay, system equipment and piping areas.

CLARIFICATION AND FILTRATION

Design Considerations

As shown in Table 1, suspended solids vary substantially on a seasonal basis with concentration levels generally lower than 50 mg/l in winter months. Much higher suspended solids concentration levels occur during spring run-offs and during the months of higher precipitation (June to September), periods when concentrations can exceed 2000 mg/l with the highest recorded level being 7,000 mg/l.

The seven year record on suspended solids (Table 1) shows that an average of 19 percent of suspended solids are smaller than 2 microns. It was decided that a solids contact type clarifier would be the most effective unit to handle water containing high suspended solids caused by either spring run-off, heavy rain falls or soil erosion.

TABLE 1

Maximum Monthly Concentration
Peace River - Suspended Solids

	1969	1970	1982	1981	1980	1979	1978
Jan.	15	25	N/A	N/A	N/A	N/A	N/A
Feb.	25	64	N/A	N/A	N/A	N/A	22
Mar.	40	445	N/A	N/A	N/A	N/A	27
Apr.	N/A	2616(10)	N/A	N/A	540	N/A	N/A
May	3614(5) *	792	2120(2)	5880(8) ***	1042	3124(6)	936
June	740	1776	800	889	3522(6)	1764(2)	1880(2)
July	424	207	4014(5) *	202	517	1686	1408(1)
Aug	338	95	4442(4) *	335	150	N/A	101
Sept.	675	24	N/A	46	1134	30	44
Oct.	108	26	N/A	34	161	48	N/A
Dec.	[illegible]	22	N/A	N/A	N/A	N/A	41
Solid Size (2/u)	21.6%	19.6%	17.0%	11.6%	21.1%	23%	N/A
# Days S.S. 2000	(5)	(10)	(11)	(8)	(6)	(6)	(3)

Note 1: Maximum monthly concentration based on average value of 5 days of highest s.s. concentration for the month.

* Indicates one day 5000ppm s.s. 6900 ppm

*** Indicates one day s.s. 7000

The seven year record on suspended solids (Table 1) shows that an average of 19 percent of suspended solids are smaller than 2 microns. It was decided that a solids contact type clarifier would be the most effective unit to handle water containing high suspended solids caused by either spring run-off, heavy rain falls or soil erosion.

The solids contact type clarifier is operated with a sludge blanket through which all of the water being treated is passed. The sludge blanket consists of previously formed floc maintained in a fluidized form. As the floc particles enlarge, they become too heavy to be maintained in suspension and consequently settle to the clarifier floor. The scraper moves the settled floc to the thickening zone at the centre of the clarifier where it is then removed as sludge.

The optimum operating pH range of alum clarification is between 5.5 and 7.5. Caustic is injected upstream of the filter splitter box in order to raise the pH of the clarifier effluent to between 7.5 and 8.3 and prevent corrosion. The caustic injection system is controlled by a pH monitoring system which samples the filter effluent.

This particular type of clarifier is most effective in the treatment of water with high levels of finely suspended solids.

Figure 4 is a schematic diagram of a solid contact clarifier, which is designed with three distinct zones as follows:

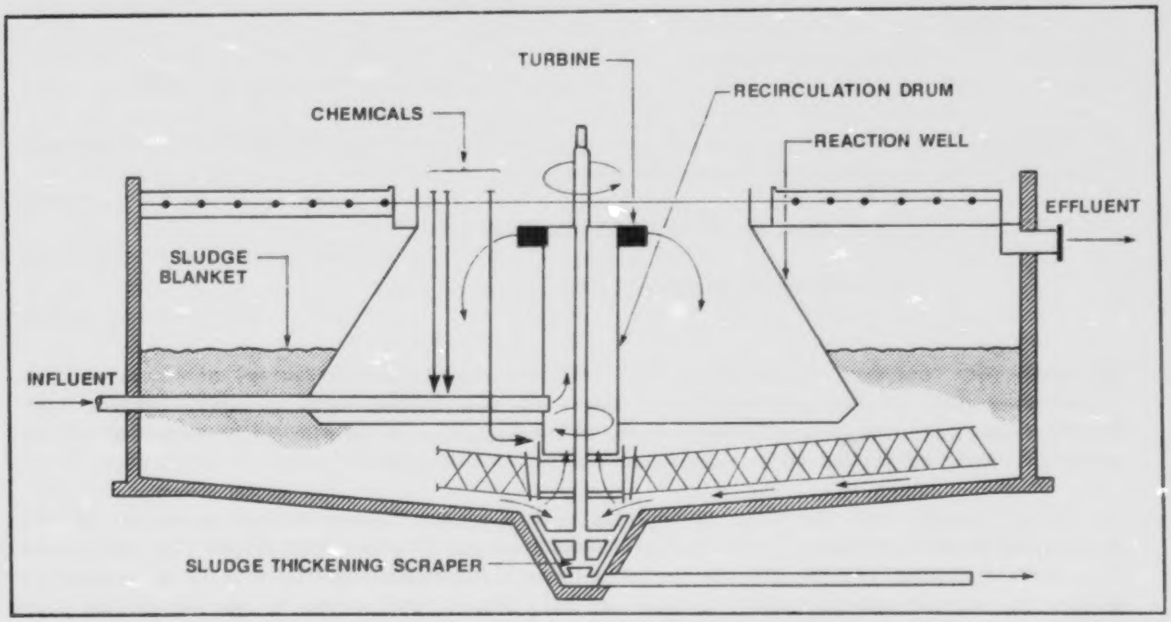
1. A central recirculation drum is the mixing zone, confined by a vertical cylindrical baffle for directing flow to the secondary zone. Within this primary zone is an axial flow mixer [turbine] for creating high rate mixing to ensure complete coagulation of the inlet water. Sludge is recirculated in this zone and mixed with incoming water, thus providing thorough interface to aid in the propagation of flocculation.

2. An annular secondary hydraulic flocculator zone contained by a conical baffle, separating it from the final stilling zone. Coagulant aid and chlorine are added in this zone to coalesce the small floc.

3. A final quiescent zone in which the water rises and flows upwards from the lower periphery of the secondary conical zone, passing through the sludge blanket. The inner annular launder at the top of the clarifier collects the water and feeds the gravity filters.

FIGURE 4

Solids Contact Clarifier



In order to accommodate the occasional high influent solids, conservative design criteria was used for the desludge line size, scraper speed and torque. A sludge scraper with 88.5 kN.m (120,000 ft/lb) torque capability was selected to ensure that heavy solids can be readily moved to the sludge cone without overtorquing. To further ensure silt removal flexibility, the scraper drive has a 3:1 variable speed drive ratio which correlates to a tip speed range of 4.8 m/min to 13.7 m/min. This speed range will ensure silt removal at times of high loading.

Desludging from the clarifier is proportional to the clarifier inlet flow rate. At a predetermined throughput, which is adjustable, the desludge operation commences with a short backflush to fluidize any heavy sludge in the cone before opening the desludge valve for a pre-set period of time.

Settling Tubes

The clarifier was designed for an initial flow rate of 7000 m^3/d and a future flow rate of 17800 m^3/d. Settling tubes will be installed in the future in order to handle the future flow rate.

The design approach used on the settling tubes was based on the fact that by providing a greater settling surface to depth ratio, settling would be speeded up. For example, a particle settling at a rate of 2 centimeters per minute would impinge on a settling surface in 5 minutes if a basin was 10 cm deep, whereas, 125 minutes would be required for particle settling in a 2.5 meter deep basin.

The clarifier was designed to operate at a maximum rise rate of 41.0 $m^3/d/m^2$ (0.7 $USGPM/ft^2$) for the initial phase and 105.5 $m^3/d/m^2$ (1.8 $USGPM/ft^2$) for the second phase with the addition of settling tubes.

Gravity Filters

Four 4880 mm (16 ft) diameter by 4880 mm (16 ft) high rate gravity filters were installed to meet the flow requirements of Phase 1 and Phase 2 with maximum hydraulic rates at 128.9 $m^3/d/m^2$ and 322.4 $m^3/d/m^2$ (2.2 $USGPM/ft^2$ and 5.5 $USGPM/ft^2$) respectively (based on 3 filters in service and 1 in backwash).

Clarified water enters the filtration system by gravity through the filter flow splitter box, which evenly divides the flow to each operating filter. After passing through the sand media the filtered water rises to the storage compartment at the top of the filter before overflowing to the filtered water storage tank. The storage compartment in the top of the filter is the source of backwash for the filter. The automatic cleaning cycle is normally initiated for each filter by a timer, however, each filter is equipped with a differential pressure indicating switch which can also be selected to initiate the backwash programme of the respective filter.

The control logic contains an interlock to prevent simultaneous backwash of more than one filter. The filter cleaning cycle includes an air scour step to assist in maintaining the filter bed in a clean condition.

In order to minimize corrosion potential, all piping from the clarifier inlet to the inlet of the vacuum deaerator is of stainless steel construction.

CHEMICAL FEED SYSTEMS

General

All of the chemical feeds to the clarifier and influent piping are injected in proportion to the inlet flow and the controlling signals are generated in the clarifier influent control system.

Chlorine

Chlorine gas is supplied from one ton containers in the chlorine storage room to the chlorine equipment room where three chlorinators regulate the chlorine gas supplied to:

1. The low lift pump bay where the gas is used to prevent biological growth in the pump bay, system equipment and piping.

2. The flocculation zone of the clarifier to oxidize any organic matters that interfere with the clarification processes.

3. The outlet zone of the deaerator to be used as a disinfecting agent for the potable water system.

The chlorine ejectors are mounted at the points of injection so that the chlorine gas is supplied under vacuum to minimize the danger of chlorine gas escaping.

Coagulant Aids

Two polymer feed systems are provided for the injection of coagulant aids, each consisting of a mixing tank, feed tank and metering pump. A third pump is supplied as a common spare.

A cationic polymer is injected upstream of the clarifier to neutralize the negative charges. Either high molecular weight, anionic or non-ionic polymers may also be injected to neutralize negative charges. These polymers are injected into the flocculation zone of the clarifier to bridge the small floc particles and cause them to agglomerate rapidly into a larger, more cohesive floc which settles quickly.

Three alternate chemical connections are provided to permit operational flexibility and ensure that the most efficient sequence of chemical addition be established.

Aluminum Sulfate (Filter Alum)

The liquid alum is stored in a 3800 mm diameter x 3660 mm high tank, with a capacity of 38 m^3 and constructed of fiberglass reinforced polyester.

There are two 100 percent duty metering pumps, (one standby) which inject the liquid alum directly into the clarifier inlet pipe.

Sodium Sulphite

The sodium sulphite feed facilities include a mixing tank complete with an agitator, and two 100 percent metering pumps used for injecting chemicals into the storage compartment of the deaerator. A residual concentration of 1 mg/l sodium sulphite will be maintained in the deaerated water storage tank and the pipeline for oxygen scavenging.

Sodium Hydroxide (Caustic Soda)

The 50 percent liquid caustic is stored in a 3000 mm diameter x 3660 mm high tank constructed of fiberglass reinforced polyester with a capacity of 24 m^3.

There are two 100 percent duty metering pumps, (one standby) which inject the concentrated caustic directly into the clarifier effluent pipe upstream of the filter splitter box. The caustic system piping is insulated and heat traced to prevent freezing since the 50 percent caustic freezes at 12°C.

VACUUM DEAERATOR

Design Considerations

The oxygen level in water plays a major role in it's corrosiveness effect. It functions as a cathodic dipolarizer at local cathodes on the surface and thereby encourages local cell action. Corrosion rates as high as 100 mils per year have been reported in steel piping handling water which is saturated with oxygen (7 mg/l to 9 mg/l).

The hot produced water and the cold source water are blended upstream of the softening system. When water containing oxygen is heated, the rate of corrosion increases rapidly. Accordingly, the oxygen content of the cold source water must be reduced to zero.

Three methods of Oxygen removal were investigated, (1) Chemical scavenging using sodium sulfite or hydrazine, (2) Counter current gas stripping, and (3) Vacuum deaeration. A study revealed that vacuum deaeration is the most practical and least costly method of oxygen removal for the PREP Source Water Supply system.

The original design intent was to install a vacuum deaerator at the central plant to process water supplied from the Peace River and Cadotte Lake. However, detailed analysis revealed that it would be more cost effective to install one deaerator at the Source Water plant and a smaller deaerator at the Central Plant to treat the Cadotte Lake supply only.

The theory of operation of a vacuum deaerator is quite simple. The solubility of a gas in a liquid is directly proportional to the pressure on the system, therefore, by reducing the system's pressure, the amount of dissolved gas will decrease. The purpose of the packing in the deaerator is merely to give the water a greater surface area, and therefore allow it to approach equilibrium conditions as it passes through the tower.

The following design criteria was specified for the deaerator:

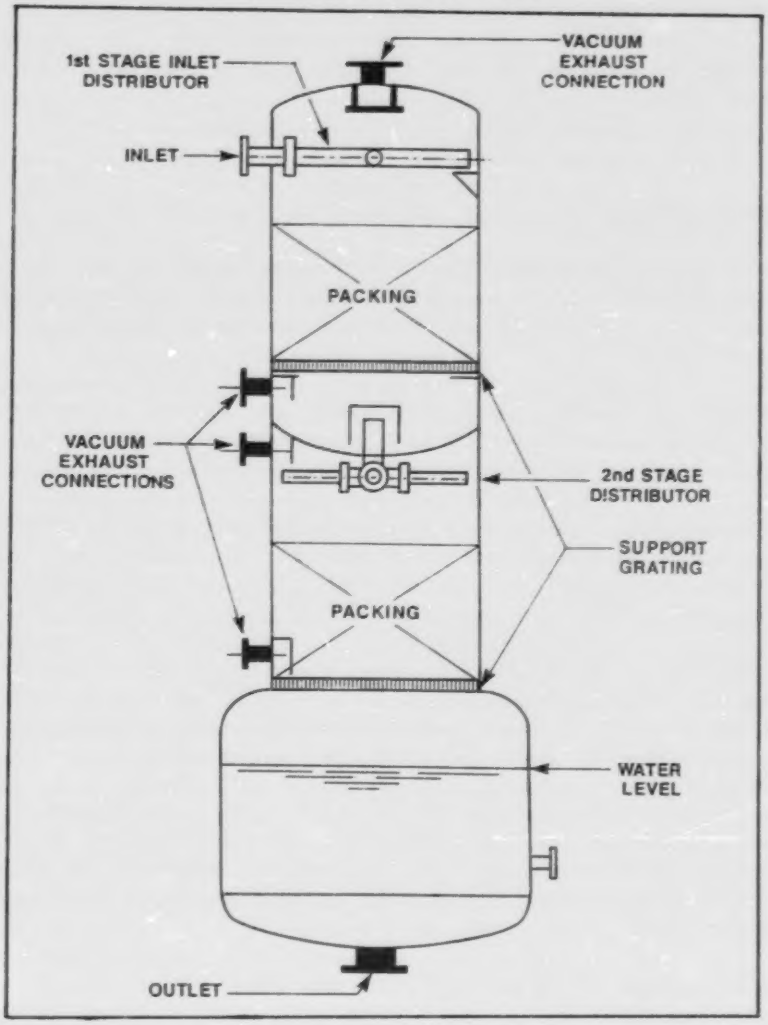
1. The vessel must be sized to prevent flooding of the packing.
2. The depth of packing must be sufficient to allow a close approach to equilibrium conditions within the unit.
3. The packing should be distributed in a manner which will minimize channeling.
4. Vapour removal lines must be sized to prevent excessive pressure drop.

The vacuum deaerator selected was a two-stage unit with 75 m^3 of integral storage, constructed of steel and having with an outlet capacity of 18,000 m^3/d (3300 USGPM).

Figure 5 is a schematic representation of the internal assembly of the deaerator.

FIGURE 5

Two-Stage Vacuum Deaerator


VACUUM
EXHAUST
CONNECTION
1st STAGE INLET
DISTRIBUTOR
INLET
PACKING
VACUUM
EXHAUST
CONNECTIONS
2nd STAGE
DISTRIBUTOR
SUPPORT
GRATING
PACKING
WATER
LEVEL
OUTLET

Two 100 percent liquid ring vacuum pumps were designed for the following tower operating conditions.

	First Stage	Second
Evacuation SM^3/H (scfm)	476 (280)	476 (280)
Pressure mm Hg ABS	47.7	10.2
Water Vapour kg/hr (lbs/hr)	3.04 (6.7)	3.04 (6.7)
Non condensable gasses		
kg/h (lbs/hr)	34.5 (76)	34.5 (7.5)

WATER STORAGE AND TRANSFER

The deaerated water storage tank, which is 14.6m diameter by 12.2m high, is sized to provide a working capacity of 60 minutes at the ultimate pumping rate of 3000 m^3/hr. The tank is designed in accordance with the American Petroleum Institute Standard 650 for welded steel tanks and is equipped with a floating roof. In order to ensure trouble free winter operation of the deaerated water storage tank, two design features are incorporated including external tank insulation and the provision for the addition of a tank heater in the future.

The tank roof deck is constructed of a polyurethane core enclosed with Spetabon aluminum, with an insulation factor equivalent to R20. This floating roof will prevent ingress of oxygen into the treated water and eliminate the need for inert gas blanketing for the storage tank.

The treated water is transferred to the Central Plant by the high lift pumps. The pumps are designed to operate initially at 327m of total dynamic head and to be modified to produce a higher head for later phases with a higher pipeline throughout.

The buried 16 inch Source Water Pipeline, which has a total length of about 25 km, climbs the escarpment which confines the river valley and then crosses 23 kilometers of slightly undulating farmland, bush and occasional muskeg before entering the western side of the Central Plant.

The pipeline was constructed on schedule, with testing, purging, and all construction work completed within a two month period in the late fall of 1985.

SCALE AND CORROSION INHIBITORS

Robert S. Spon
Technical Director
Kjell Water Consultants, Inc.

Scale and corrosion inhibitors classified as "polyphosphate" were first recognized by a U.S. patent in 1887 covering the use of trisodium phosphate to prevent boiler scale. However, it wasn't until half a century later that sodium phosphates were introduced as synthetic cleaning agents in place of soap. Sodium Tripolyphosphate was used in the first commercially produced detergent by Proctor & Gamble, called TIDE. The development of the Langelier Saturation Index was closely followed in the late 1930's by a "threshold treatment" process which called for the addition of a few parts per million of sodium hexametaphosphate to water for control of the precipitation of calcium carbonate. This report by O. Rice and G.B. Hatch was expanded as they continued to emphasize the usefulness of "thresholding" to minimize corrosion and red water. It is from this initial basis, that we today have been able to formulate and incorporate many of the properties of sequestration, dispersion, and buffering into a product that can withstand the many demands that occur throughout a potable water distribution system.

To briefly describe the different phosphates that are available for use as water conditioners, let me first break them into two basic categories: orthophosphates and condensed phosphates.

The orthophosphates are the simplest sodiumphosphate salts, such as mono, di, and trisodium phosphate. These forms are produced by combining orthophosphoric acid (H_3PO_4) with soda ash (Na_2CO_3) and sodiumhydroxide ($NaOH$). These orthophosphates are then fused through a heating process to form "condensed phosphate", which can be of either the crystalline or glassy form.

The crystalline sodium tripolyphosphate and glassy sodium hexametaphosphate are two easily recognizable products widely used in the water treatment industry. The next steps of formulation under various degrees of time and temperature lead to the specialty blends of proprietary liquid compounds. These condensed phosphates contain a mixture of phosphate oxygen chains of various lengths. It is the additional step of forming linear chain and polymerized phosphates from the basic building blocks of generic polyphosphates that leads to the breakpoint of success in the somewhat marginal results of some existing phosphate treatment programs.

Phosphates used in potable water supply systems are known to effectively reduce tuberculation in distribution lines, minimize red and black water caused by high iron and manganese or the by-products of the aggressive water.

These corrosion problems are extensive in old cast iron main systems. Estimates have placed losses in supply distribution systems at $700 million each year, not including additional costs for services and inside plumbing. Corrosion of these ferrous metal pipes unfortunately results in the formation of iron oxide by-products which occupy a greater volume of space than the metal loss from the pipe wall. The accumulation of this tuberculation reduces the flow rate and overall carrying capacity of lines. One inch of scale on an eight inch water main reduces flow capacity by more than 50 percent. Increased electrical pumping costs and fire insurance ratings reflect this unfortunate consequence of even a minor scale or corrosion problem. These deposits are also quite evident through excessive replacement of hot water systems where the reaction rate of scale formation is accelerated. In addition to these extreme replacement costs, taste, color, and odor from iron and copper corrosion (red and blue-green staining) are potential sources of consumer complaints.

Through use of polymerized phosphate, inhibition of these problems is both efficient and effective. The example of the scale formation or the corrosion rates in hot water systems can be evidence of how the phosphate treatment is performing in the entire system. Generally, higher temperatures cause more rapid rates in chemical reactions and usually reduce the solubility of minerals, as seen by calcium carbonate scale formation in heat transfer surfaces. Except for cases where the alkilinity of the water is less than 50 ppm, the solubility product for $CaCO_3$ decreases. This is due to the buffering capacity of the higher bicarbonate concentration and its hindrance of the temperature effect on pH.

These basic principles can be correlated to the characteristics of phosphates in sequestering iron and conditioning the hardness while under the extreme condition of a hot water tank at temperatures of 130 to 160 degrees Fahrenheit. Just by concentrating on the simple activities in a hot water heater we can save ourselves many months of testing and theorizing about how effective the phosphate program is within the entire distribution system. If a phosphate can stand up to the stringent requirements in the hot water system, we can be very optimistic about satisfying the water utility and the water user.

After a complete water analysis has been performed for the specific components, (iron, manganese, silica, alkalinity, hardness (calcium, magnesium), and pH) the chemical distributor and supplier of the phosphate treatment should verify and dose the product based on the water quality parameters. After the background tests and standards are compiled, the phosphate can now be monitored to show the overall performance. Test data from field analysis for ferrous and ferric iron, phosphate (PO_4 and total), manganese, turbidity, free and total chlorine are all valuable pieces of information. Various sites in the system can be determined for control and experimental samples. These tests should be performed at the water well or treatment plant source, fire hydrants (before and after flushing), residential water spigots and taps (inside and out), low flow areas, high consumption industrial accounts, schools, hospitals, hot and cold, new and old services alike. Remember, that the more information available from the beginning, leaves less room for error down the road.

In order to verify dosage feed rate at the well head or treatment plant, the poundage or gallons of actual solution injected at the source can be compared to the parts per million observed at these sampling points in the system.

A very stable phosphate will give consistent test readings from the source of application to the very end of the system, regardless of flow rate and temperature of sample. A decrease in phosphate content shows that the product is precipitating and reverting to the ortho state that we described earlier when discussing the manufacturing and refining process. In this ortho state iron and manganese cannot be sequestered, thereby allowing discolored water to start reoccurring out in the lines. Therefore, a product with low reversion to ortho and stable properties is desired for use with high mineral content water.

This potential for reversion from complex to ortho can actually be seen by using the hot water tank comparison. The difference between the ortho going in on the cold side and the level on the hot side gives the percentage of breakdown through increased temperature. Comparing this amount reverted with the total phosphate available in the compound also shows stability to remain in the active state. Quite often they will be totally broken down before the hot water heater and will give lower readings on the hot side due to scaling out of chemical along with the hardness. Certain generic polyphosphates are known to revert several times faster than the formulated linear chain compound. In a system with low flow areas, dead-end lines, process water use, and in domestic hot water tanks this break-down doesn't allow for complete relief of the problem. Therefore, testing phosphate reversion with a simple field test kit can give valuable information to the utility when comparing products. A material with low stability is not very cost effective. Consistent field test readings of phosphate and iron show stability.

Once the objective to clear up red water or reduce corrosion or clean-up tuberculation has been defined, it can be very easy to form trends and project the success rate through "common sense use" of test data. Anyone who has programmed or even used a computer can relate to the statement "garbage in - garbage out". A potable water system is much like a computer. The hardware can be considered as the mechanical equipment; the well pumps, treatment plant, filters, valves, water mains, service lines, etc. The software would be the chemical treatment, such as lime, alum, polymer, chlorine, fluoride, or phosphate. The relationship or the interface between these two mechanisms is very critical and quite often the key to success or failure. The value of one can only be determined in combination with the other. Therefore, I want to stress the significance of this example especially for our discussion about phosphate treatment.

The phosphate program is only as good as your means of evaluation, whether it simply be customer complaint log sheets, good and rapid clean-up of hydrant flushes, or thorough laboratory studies for water flow C-factors and corrosion rates. Be it kept simple or made complex, we will all eventually hear about the success rate from the customer. Phosphate treatment programs are subjective. Unless you have been involved with the original stages of passivation through clean-up and maintenance, it is difficult to describe the satisfaction receive when "dirty water" problems have been minimized or eliminated after years of increasing customer dissatisfaction. Phosphate programs are not like lime softening, or chlorination, or fluoridation where the process either works or it doesn't, the chemical residuals present or absent.

The phosphate treatment program is relative to the severity of the problem, and dependent on the effectiveness of the prescribed additive.

Today, through the vast amount of background in field studies available for comparison, it is now possible to expect absolute results using linear chain polymerized phosphates where other phosphates have fallen short in the past. There's no need to rely only on partial success. As a challenge, let some of the previously described ideas and procedures assist you in obtaining the optimum effectiveness of scale and corrosion inhibition form use of the linear chain phosphate in a complete water treatment program.

TRIHALOMETHANE FORMATION PREVENTION AND COLOR REMOVAL WITH OZONE

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INTRODUCTION

Ozone is a strong oxidant, second only to fluorine in oxidizing potential. The word, "ozone," derived from the Greek "Ozein" - to smell, is used to identify ozone because of its pungent odor.

In nature, ozone can be found and easily recognized in the air after thunder or lightening storms. At higher levels of the atmosphere, ozone is formed photo chemically by the action of ultraviolet rays on oxygen. This reaction acts to shield the earth from potentially hazardous UV rays.

Ozone is a triatomic form of oxygen. Though naturally occurring, ozone is very unstable. Ozone has a short half life (15 minutes), and decomposes readily to the common diatomic form of oxygen. These characteristics of ozone make storage difficult, so for commercial application ozone is generated at point of use.

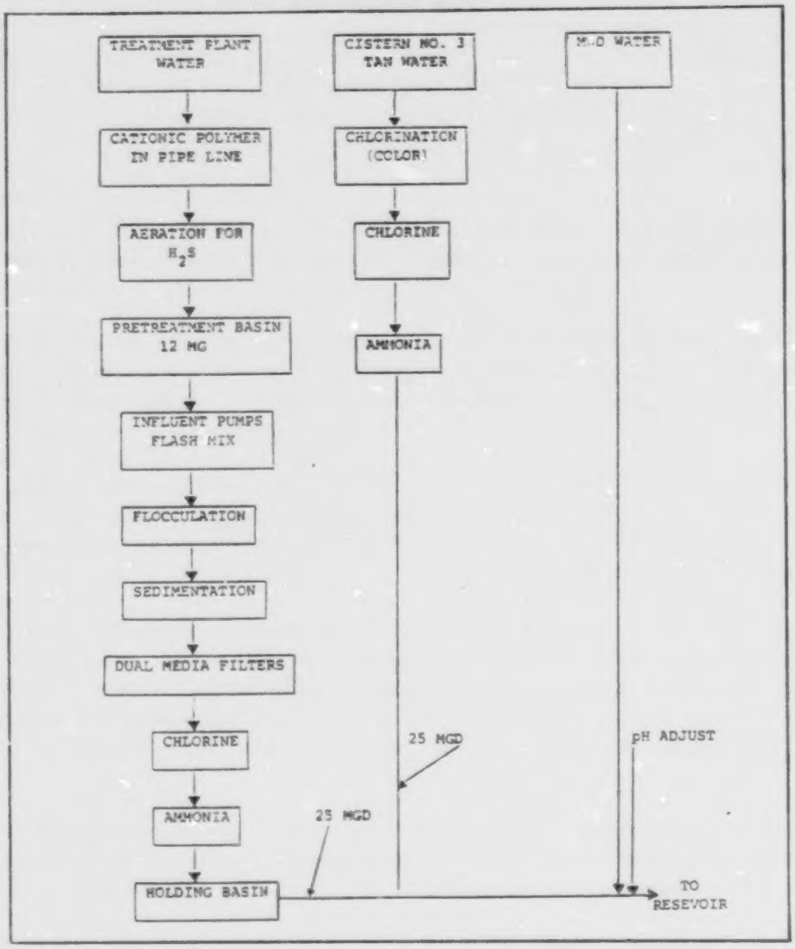
Commercially, ozone is typically produced by silent electric discharge through air or oxygen. The electric discharge takes place in a gas space between two electrodes which are separated by a dielectric. The electrodes are supplied with an alternating current. Ozone production is an endothermic process, with only a small part of the required energy actually producing ozone. The rest of this energy is released as heat, which must be dissipated, typically through cooling water.

A pilot study utilizing ozone at the City of Long Beach, California Water Treatment Plant has been completed. The scope of the study was to determine the effectiveness of ozone in treatment of water for color removal and trihalomethane (THM) formation prevention.

Long Beach Water Treatment Plant

The City of Long Beach Water Treatment Plant is capable of treating 7885 m^3hr^{-1} (50 million gallons per day). The water is pumped from deep wells. The location of the wells is such, that as water is removed from the aquifer, it is contaminated with color bodies and various other organics. The current water plant uses chemical coagulation with polymers to remove color bodies. Multi media filters are used to remove suspended particles. Chlorination is used for disinfection, followed by ammoniation for THM control (see Figure 1). The current design adequately addresses existing water quality standards, but may have to be modified to meet more stringent standards of the near future. Of particular importance to water quality for the City of Long Beach is THM formation prevention. Currently, 40 ug L^{-1} of TTHMs are produced by the plant.

FIGURE 1
Long Beach Process Flow


TREATMENT PLANT WATER
CATIONIC POLYMER IN PIPE LINE
AERATION FOR H_2S
PRETREATMENT BASIN 12 MG
INFLUENT PUMPS FLASH MIX
FLOCCULATION
SEDIMENTATION
DUAL MEDIA FILTERS
CHLORINE
AMMONIA
HOLDING BASIN
CISTERN NO. 3 TAN WATER
CHLORINATION (COLOR)
CHLORINE
AMMONIA
MWD WATER
25 MGD
25 MGD
pH ADJUST
TO RESEVOIR

Trihalomethanes

The recently enacted Revised Safe Drinking Water Act of 1986 has mandated that the Environmental Protection Agency promulgate standards for total trihalomethane levels in municipal water supplies. The current regulations specify a maximum contaminant level for THMs of 100 ug l^{-1} but will be much lower in the future. Further modifications than the use of chloramine will be needed to reach lower THM levels, including the elimination of free chlorine in the water treatment process.

Ozone

In the continuing effort to provide its citizens with the safest and most palatable water, the City of Long Beach has undertaken a pilot study to investigate the use of ozone for color removal and THM formation prevention. Ozone is one of the most studied chemicals as an alternative to chlorine for water treatment. As a strong oxidizing agent, ozone, like chlorine reacts with organic constituents in water. Ozone, unlike chlorine, does not form THM by-products. The major disadvantage of ozone is that it is not persistent in a water solution, therefore not effective alone in controlling the heterotrophic bacterial population. Ozone has been used in water treatment in Europe for over 75 years. In 1975, there were 5 water plants in the U.S that used ozone, in 1987, there are approximately 30 plants using or implementing ozone.

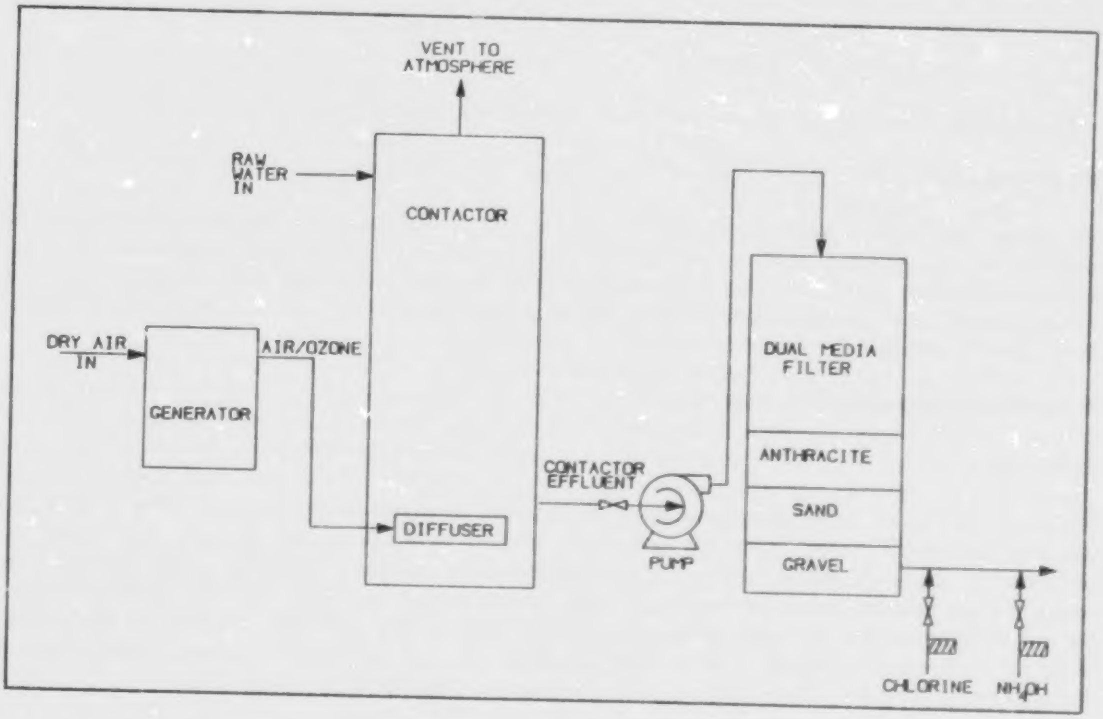
EXPERIMENTAL APPARATUS

Pilot Plant

A small scale pilot plant was built to study the effectiveness of ozone for color removal and THM formation prevention while duplicating the major characteristics of the existing Long Beach Water Plant. A schematic of the pilot plant is shown in Figure 2.

FIGURE 2

Pilot Water Treatment Plant


VENT TO ATMOSPHERE
RAW WATER IN
CONTACTOR
DRY AIR IN
GENERATOR
AIR/OZONE
DIFFUSER
CONTACTOR EFFLUENT
PUMP
DUAL MEDIA FILTER
ANTHRACITE
SAND
GRAVEL
CHLORINE
NH_4OH

Ozone Generator

The ozone generator was a Capital Controls model GL-1, with a capacity of 19 gm hr^{-1} (1 lb. per day) at a concentration of 2% ozone by weight in air, using a dry air feed.

Contact Chamber

The contact chamber was a 365 cm (12 ft) vertical pvc pipe, 30.4 cm (12 in.) diameter, with a 2.54 cm (1 in.) inlet at the top, and a 5.08 cm (2 in.) outlet at the bottom. A gate valve was fitted on the outlet pipe to regulate the depth of water within the chamber. The ozone-air mixture was injected through a dispersion stone near the bottom of the chamber, so that a counter current gas-liquid transfer system was achieved. The contact chamber was vented to atmosphere through a pipe at the top of the chamber.

Dual Media Filter

The dual media filter was constructed of plexiglass. Inside dimensions were 53.3 cm (21 in.) wide x 53.3 cm (21 in.) deep x 221 cm (87 in.) high. The filter media, from the bottom consisted of:

15 cm (6 in.) gravel of size 3.8 - 2 cm (1 1/2 - 3/4 in.)
5 cm (2 in.) gravel of size 2 - 1 cm (3/4 - 3/8 in.)
5 cm (2 in.) gravel of size 1 - .6 cm (3/8 - 1/4 in)
5 cm (2 in.) gravel of size .6 - .3 cm (1/4 - 1/8 in.)
20 cm (8 in.) sand effective size 0.45 - 0.55 mm
51 cm (20 in.) anthracite

The operating liquid level was 102 cm (40 in.) above the anthracite.

Chloramination

Following the filter were inlets for chlorine (Cl_2) and ammonia (NH_4OH) injection. The combination of the pipe diameter and distance between the inlets allowed a 24 second interval between chlorine injection and ammonia injection. This duplicates the 24 second interval between chlorine and ammonia injection in the Long Beach Water Plant at the average flow rate of 3942 M^3 hr^{-1} (25 MGD).

EXPERIMENTAL PROCEDURES AND RESULTS

Color Removal

The first series of experiments was designed to observe the removal of color. The pilot plant was operated until steady state was achieved. Samples were taken from the raw water inlet, and effluent of the dual media filter. These samples were analyzed for color and turbidity. The ozone dosage applied was reported as average concentration of ozone in contact chamber water before reaction. The residual ozone concentration was measured at the effluent of the contact chamber. Ozone in liquid samples was analyzed by the iodometric method, quantified by thiosulfate titration. These results are presented in tabular form in Table 1.

TABLE 1
Oxidation of Color by Ozone

Sample*	Color (color units)	Turbidity NTU	Water Flow M^3hr^{-1}03 (gpm)	Applied mg L^{-1}	O_3 Residual mg L^{-1}
R	40	0.43	2.6		
E	4	0.19	(11.5)	4.5	0.4
R	40	0 32	2.7		
E	4	0 23	(11.8)	4.0	0 4
R	32	0.48	2.6		
E	0	0.24	(11.5)	4.0	0 4
R	38	0.75	2.2		
E	1	0.18	(9.6)	5.1	0.6
R	38	0.75	2.2		
E	1	0.18	(9.8)	4.3	0.5
R	57	1.12	2 1		
E	3	0.30	(9.4)	5.5	0.1

* R was raw water source E was effluent from dual media filter

THM Formation Prevention

The second series of experiments was designed to observe levels of total trihalomethanes in ozonated water Raw water was ozonated at 5 mg L^{-1} ozone in the contact chamber and filtered through the dual media filter. The filter effluent was chloraminated. The chlorine was added first, followed by ammonia addition 24 seconds later (downstream) at a weight ratio of chlorine to ammonia of 4.5 to 1. The pilot plant was operated at steady state and samples were taken of raw water inlet and contactor effluent. These were analyzed for color. Samples were taken of the pilot plant effluent and analyzed for total trihalomethanes. The results are presented in tabular form in Table 2. The high TTHM values obtained in the final two runs of Table 2 reflect the high initial color and precursor concentration in the raw water.

TABLE 2

Raw Water color color units	Contactor effluent color color units	Chlorine mg L^{-1}	TTHM ug L^{-1}	Standing Time before TTHM analysis
38	1	1.5	1.9	24 hr
38	1	1 6	2.1	24 hr
38	1	3 0	3.2	24 hr
36	1	2.6	4.0	3 min.
38	1	2.6	5 0	24 hr
57	3	2.6	11.0	24 hr
57	3	2.8	11.0	24 hr

A variation of the above experiment was implemented by reversing the order of chlorine and ammonia addition. All other parameters were the same. The change reduces the time in which trihalomethanes can form. These results are presented in tabular form in Table 3.

TABLE 3

Raw Water color color units	Contactor effluent color color units	Chlorine mg L^{-1}	TTHM ug l^{-1}	Standing Time before TTHM analysis
38	1	2.1	1.8	24 hr
38	1	4.2	1.3	24 hr

CONCLUSIONS

The first series of experiments demonstrate ozone was effective in reducing color in Long Beach Water Plant raw water. Organic color was reduced from raw water levels of 32 to 57 color units to 1 to 4 color units in ozone treated water.

The second series of experiments demonstrate that the replacement of chlorine by ozone in the pretreatment of raw water reduces significantly the level of TTHM in the effluent Raw water (organic 38 color units) ozonated, filtered, and chloraminated exhibited a TTHM level of 5 ug L^{-1} or less. When the raw water color level was high (57 color units) a maximum of 11 ug L^{-1} was observed. Reversing the order of chlorine and ammonia addition from the normal procedure is apparently quite effective at reducing THM formation. This procedure resulted in less than 2 ug L^{-1} TTHM for raw water of initial organic color 38 color units.

CAST IRON WATERMAIN CORROSION

EDMONTON PLANS FOR THE FUTURE

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INTRODUCTION AND BACKGROUND

The City of Edmonton s water distribution system includes approximately 2600 km of watermains that supply 280,000 services and a population of about 571,000 people within the city limits. The City also supplies water to a further 184,000 regional users. The watermain pipe materials are mainly asbestos cement, polyvinyl chloride (PVC), and cast iron In 1986, there were approximately 1100 km of cast iron pipe in service within an approximate overall area of 3600 hectares. The extent of cast iron watermains peaked at approximately 1200 km in 1966, but the large number of pipe breaks since then has necessitated the replacement of about 100 km of the cast iron with PVC pipe.

Breaks in the remaining 1100 km of cast iron watermains continue to be a significant problem in Edmonton.

Although the cast iron system comprises only about 40 percent of the total length of watermains in service, it accounts for over 90 percent of the city's total watermain breaks. There were a total of 1,688 breaks in 1985, of which, as shown in Table 1 1,629 occurred in the cast iron system. Table 1 also shows the recent history of watermain breaks maintenance, and renewals in Edmonton Although the number of breaks occurring from year to year is inconsistent, the trend does show an increase The city has assumed that this increasing trend indicates steady deterioration of the system and has adopted a long-range plan to renew all cast iron mains in a 55-year period at the average rate of 20 km per year at an approximate cost of $7.5 million per year in 1986 dollars.

TABLE 1

Recent History of Cast Iron Watermain Maintenance in Edmonton

Year	1981	1982	1983	1984	1985	1986
Watermain Breaks	968	1351	1026	1130	1629	1100
Repair Costs[a]	$2 42m	$3.38m	$2.57m	$2.83m	$4.07m	$2 75m
Renewals, Km	4.8	5.6	1.3	10.4	20.8	21.3
Renewals, Costs[b]	$1.58m	$2 28m	$0.53m	$4.09m	$8.84m	$8.0m

[a] Estimated based on $2 500 per repair (1986 cost)
[b] In year of occurrence dollars x 1000
[c] Estimated

CAUSES OF BREAKS

Studies showed that nearly all cast iron pipe in the city has been weakened to some degree by external corrosion. The form of corrosion is graphitization--the selective dissolution of iron from the pipe wall--which leaves weak plugs of carbon where sound metal once was. The graphitization is due to a number of factors:

- Low soil resistivity
- Minerals [sulphate in particular]
- Clay soil type

These factors are primarily functions of the geotechnical environment occurring at pipe depth. The dominant factor is generally considered to be the soil resistivity which is a measure of the resistance of the soil to conducting electrical energy.

A commonly used relationship between resistivity and the severity of corrosion is shown in Table 2

TABLE 2

Resistivity and Severity of Attack

Soil Resistivity [Ohms-cm]	Severity of Corrosion Attack
<500	Extremely corrosive
500 - 1,000	Severely corrosive
1 000 - 2,000	Corrosive
2,000 - 5 000	Moderately corrosive
5,000 - 10,000	Mildly corrosive

The predominant soil type at pipe depth is a silty clay which typically has a low resistivity. Perched water tables and capillary effects on the groundwater table are likely to increase the moisture content at pipe depth, further lowering the resistivity. The overall effect is that most of the soils at pipe depth in Edmonton are very corrosive when considered in terms of resistivity.

Other factors that can add to the corrosion rate are galvanic [corrosive] effects from buried copper services and electrical grounds and microbial corrosive effects from anaerobic sulphate-reducing bacteria. Copper grounds are in contact with the watermains via the connection of the electrical power system neutral to water pipes in houses as required by the Electrical Code. Anaerobic sulphate-reducing bacteria accelerate corrosion of cast iron through the release of corrosive wastes by their metabolic removal of oxygen from sulphate compounds These bacteria are common in sulphate-rich clay soil

To date, internal corrosion of the piping system has not been considered as having contributed significantly to the overall break rate.

The result of all these corrosive factors is that Edmonton s cast iron watermains are all graphitized to some degree. Soil corrosive conditions vary greatly over small distances (metres) and are characterized by localized "hot spots" within otherwise consistent areas. The entire cast iron service area therefore, can be considered highly corrosive based on commonly accepted criteria

The most graphitized pipes are not necessarily those with the highest break densities since forces inside and outside the pipe essentially control the break rate. Generally, the breaks will increase as the cast iron pipe becomes weaker from progressive corrosion. The corrosion rate will not be reduced as graphitization progresses because the carbon residue is not a protective corrosion product. In addition to the variation in forces acting on the pipes at any point, the variables of pipe metallurgy and construction quality also influence break rates.

EXISTING SYSTEM FOR REPLACING MAINS

Records of watermain breaks have been kept in Edmonton for over 30 years. These records include details of where the break occurred, the size and type of pipe, the type of joint, the condition of the pipe, and the apparent cause of the break. All those factors are summarized annually and the results compared to assess variations on a year-to-year basis. Until 1985, there were no fixed criteria in place to determine when or how much pipe should be replaced and, as shown previously in Table 1, the amount varied from year to year. In 1985, however, it was recognized that as part of the multi-year replacement program, some method was needed to determine in a consistent manner which pipe was most in need of replacement and what length of each pipe should be replaced.

The method which was adopted is as follows

The break reports for cast iron watermains are compiled monthly, and the locations of the breaks are recorded on a map of the system. The number of breaks for each span of pipe between valves is added to the number of breaks that have occurred on that same span of pipe during the preceding 4 years Thus, a "watermain" is defined as the span of pipeline between valves or other suitable reference points, and a 5-year rolling break history is maintained for each watermain. At the end of each calendar year the breaks are totalled for each defined watermain and the break frequency calculated.

$$\text{Breaks/km/year} = \frac{\text{Number of breaks in 5 years}}{\text{5 years x watermain length in km}}$$

The watermains are then ranked in order of decreasing break frequency. Those with break frequencies greater than five breaks/km/year are designated as candidates for renewal during the next construction season subject to available funds. The format of the list of renewal candidates for 1986 is shown in Table 3.

In 1986, 115 watermains totalling 26.5 km were candidates for renewal. Based on the previous year's costs, a target list is then selected. As shown in Table 3, the 1986 target was to renew 22,448 m of pipe.

TABLE 3

Format of 1986 Watermain Renewal Candidates

WARD	RANK	LOCATION OF MAIN TO BE REPLACED	LENGTH IN METRES	NO. OF BREAKS PER ANNUM 81	82	83	84	85	TOTAL	NO. OF BREAKS PER KM/YEAR	COMMENTS
1	1	114 AVE. BETWEEN 156 STR. AND 157 STR.	113	0	0	0	0	8	8	14.16 **	
1	2	LW 158 STR. BETWEEN LW 101 * AVE. AND 102 AVE.	173	1	5	1	1	4	12	13.87	* Stony Plain Road
3	3	LS 119 AVE. BETWEEN LE 77 STR. AND LE 76 STR.	114	0	1	1	4	1	7	12.28	
2	4	122 B AVE. BETWEEN 129 A STR. AND 131 STR.	288	0	5	2	5	5	17	11.81	
1	5	LW 152 STR. BETWEEN 104 AVE. AND 105 AVE.	171	5	0	0	3	2	10	11.70	
3	6	LN 127 AVE. BETWEEN 68 STR. AND 70 STR.	297	1	2	3	1	10	17	11.45	
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2	112	LW 114 STR. BETWEEN LW 130 AVE. AND 132 AVE.	195	0	0	0	1	4	5	5.13	
4	113	111 STR. BETWEEN 102 AVE. AND 103 AVE.	275	1	0	0	1	5	7	5.09 **	Incl. 103 Ave., 111-112 Str.
6	114	LN 77 AVE. BETWEEN 87 STR. AND 89 STR.	200	0	3	1	1	0	5	5.00	
1	115	165 STR. BETWEEN N 93 AVE. AND 95 AVE.	160	0	2	1	1	0	4	5.00	

TOTAL LENGTH OF REPLACEMENTS:	26535
DATE OF ISSUE:	February 14, 1986
REVISION DATES:	February 17, 20, 21, 24, March 6, 1986
LATEST REVISION:	March 19, 1986
ISSUED BY:	Water Network Engineering Section Edmonton Water and Sanitation

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** Commercial or Industrial

A summary of the 1986 renewal candidates is presented in Table 4. This table highlights that although the renewal candidates comprised only about 2 percent of the total cast iron watermain system, they incurred between 14 and 22 percent of the breaks during the preceding 5 years.

TABLE 4

Break History of 1986 Renewal Candidates (26.5 km or 2.3% of total installed length)

Year of Record	1981	1982	1983	1984	1985
Breaks	126	183	157	164	354
Break Density (B/km)	4.75	6.91	5.92	6.19	13.36
Remainder of Cast Iron Pipe at Year End					
Length, km	1185	1179	1178	1168	1147
Breaks	780	1168	862	965	1228
Break Density (B/km)	0.66	0.99	0.73	0.83	1.07
Ratio of Breaks by 1986 Renewals Candidates to Total CI Breaks	14%	14%	15%	15%	22%*

*Break density criteria introduced.

The renewal criterion of five breaks/km/year was somewhat arbitrary, based on available funds and a desired level of customer service. Part of the study[1] on which this paper is based was to assess the suitability of that criterion.

BREAK PROJECTIONS AND MITIGATIONS

In order to assess alternative strategies for dealing with the watermain break problem, costs associated with these strategies must be developed and compared Key assessments in such an analysis are how break rates will vary in future years and what mitigative methods can be developed to reduce or eliminate the breaks.

BREAK PROJECTIONS

To enable costs of future break repairs to be estimated, break projections in the cast iron watermain system were developed.

The city's watermain break records were analyzed using a format developed by the University of Alberta[2]. Approximately 12,000 break records for the period 1972 to 1984 were stored on computer magnetic disks.

The break records were analyzed by dividing the city into small areas using a grid 35 arbitrary units wide (east-west) and 25 units high (north-south). An example of part of the city with the grid superimposed is shown in Figure 1 The city's break record data were tabulated for each grid cell by year and used to calculate a straight-line curve by linear regression. The purpose of the curve was to predict future watermain breaks based upon the history of breaks in each grid cell. This procedure was also used by CH2M HILL in a similar study in Winnipeg[3]. In that study alternative forecast methods included either linear regression or double-log transformation, time series trend analysis or simple annual averages. In Edmonton, the above alternatives were reviewed and compared to the linear regression slopes and intercepts used in the U of A analysis. It was concluded that, although there was considerable scatter in the data, the original linear regression values were sufficiently accurate for the study. CH2M HILL then extended the original work by analyzing break densities rather than simply the number of breaks. A plot of a typical grid cell break record with the calculated regression line is shown in Figure 2.

The length of cast iron watermain in each grid was estimated. These lengths were then combined with the number of breaks occurring within each grid cell to obtain the break density for input to the predictive model. A microcomputer data base was then prepared using the pipe inventory and regression curves to produce a predictive model of the break density for each grid cell in any given year. The results of this analysis are shown in Table 5.

TABLE 5

Distribution of Break Density

Break Density 1986 to 2021 (breaks/km/year)	Number of Grid Cells	km of Cast Iron Pipe	Percent of Total km of Cast Iron Pipe
greater than 5.00	65	114	10
2.00 to 4 99	80	114	10
1.00 to 1.99	66	125	10
less than 1.00	664	788	70
Total	875	1141	100

FIGURE 1

Typical Grid Overlay

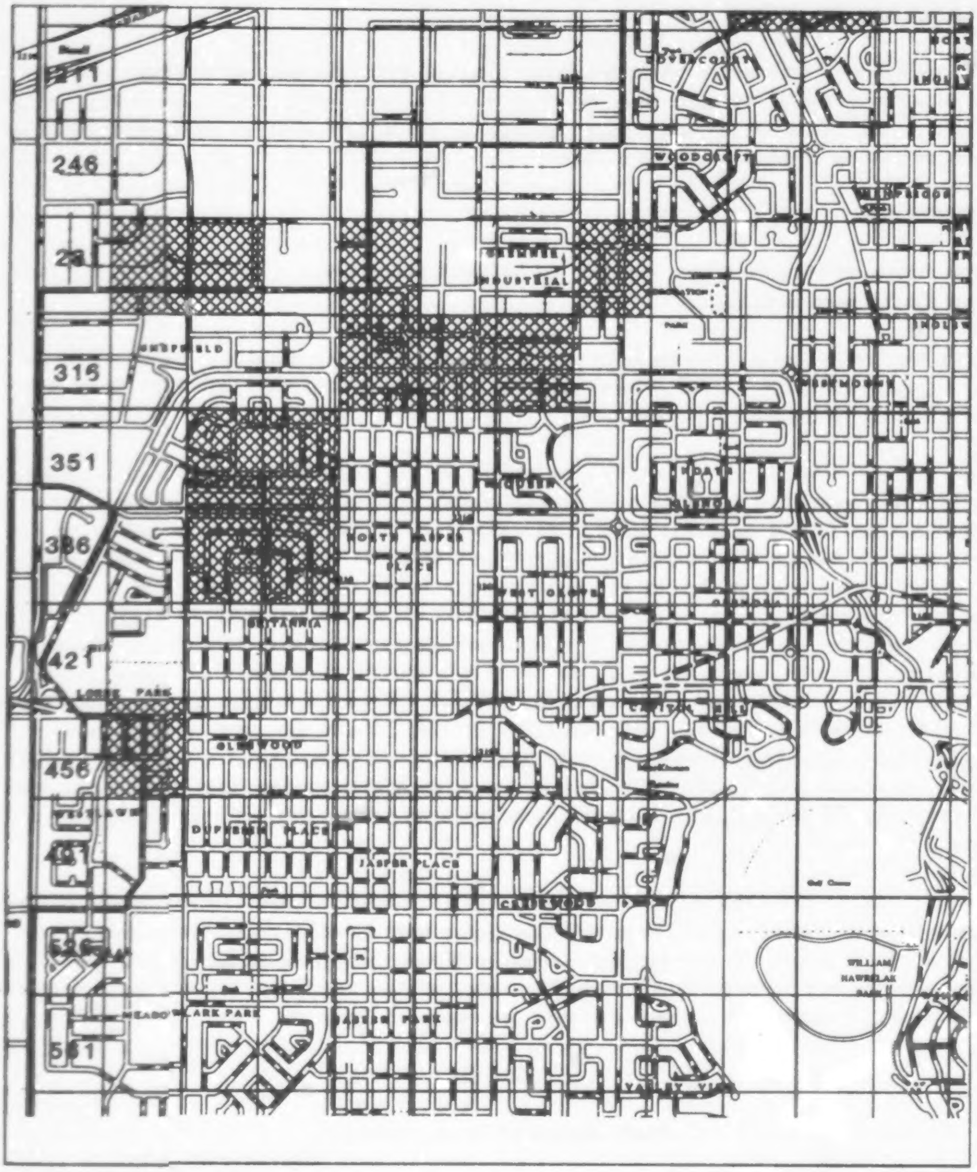
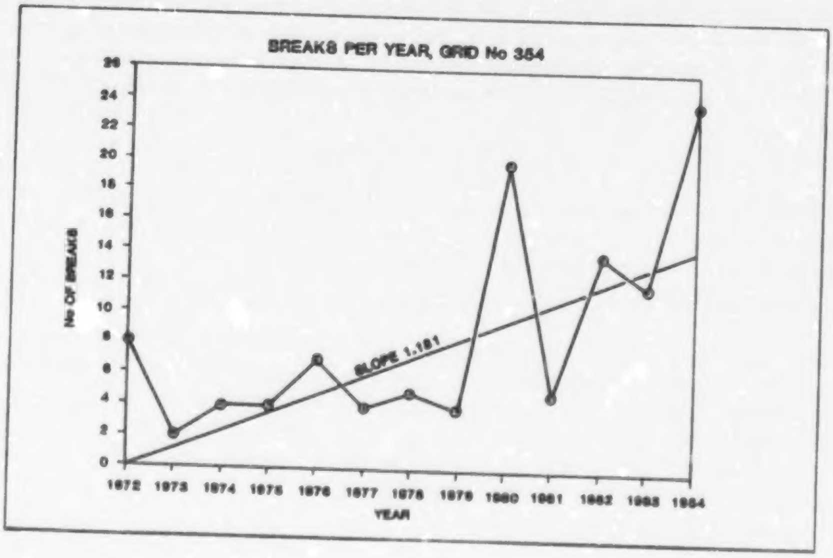

211
246
281
316
351
386
421
456
491
526
561
WOODCROFT
WESTMOUNT
INDUSTRIAL
GLENORA
NORTH JASPER PLACE
WEST GROVE
BRITANNIA
GLENWOOD
WESTLAWN
DUFFERIN PLACE
JASPER PLACE
CRESTWOOD
MEADOWLARK PARK
VALLEY VIEW
WILLIAM HAWRELAK PARK

FIGURE 2

Typical Grid Cell Break Record



These results indicate that, in general, the cast iron pipes with the highest break densities are concentrated in relatively small areas of the cast iron watermain system. For example, mains having break densities greater than five breaks/km/year represent about 10 percent of the total length of cast iron pipe. Most of the cast iron system has a relatively low break density. The implication is that while renewal is an appropriate measure in areas of high projected break density, break prevention techniques of cathodic protection or cement mortar lining might be useful in the remaining large areas of the city if they are technically and economically practical.

MITIGATION BY CEMENT MORTAR LINING

Cement mortar lining of pipe in place according or AWWA Standard C602 is a common method of rehabilitating cast iron watermains. The method involves dewatering the main to be lined, cleaning the inside of the pipe to remove rust and scale, then applying a thin layer of cement mortar to the cleaned surface.

Historically, the process has been directed at preventing the recurrence of rust tubercles inside the pipe and at restoring capacity and hydraulic characteristics of cast iron pipe. The failure mechanism in most of Edmonton s cast iron watermain leaks, however, is usually breakage or blowout caused by the action of underground forces on weakened pipe. If the lining could be used to strengthen the pipe in place while preventing internal corrosion cast iron watermain leaks would be reduced.

The ability of cement mortar lining to add structural strength to a deteriorated watermain is not well documented. Limited experience in Winnipeg, with watermain and soil conditions very similar to Edmonton, indicate from an analysis of leak trends before and after lining the lining reduced the average leak frequency by 75 to 80 percent over a 15-year period. In this instance, cement mortar lining appears quite effective at reducing leaks.

To be feasible, cement mortar lining not only must add structural strength but must not detract from pipe hydraulics and should, in fact be beneficial. The internal friction coefficients (C factors) of cast iron watermains generally decrease with time due to rusting of the iron surface. Cement mortar lining can increase the C factor sufficiently to offset the reduction in pipe diameter by the 4 to 7 mm thickness of the lining. The allowable lining thickness can be calculated using the relationship between the ratio of pipe diameters, C-values, and hydraulic slopes (S=h/L) for the same flow rates (Q) as shown in the following example.

Given:

Hazen-Williams formula

$$h = 0.002083\, L \left[\frac{100}{C}\right]^{1.85} \frac{Q^{1.85}}{D^{4.8655}}$$

Diameter of unlined pipe (D1) = 200
Design C-value of unlined pipe (C1) = 90
C-value of lined pipe (C2) = 110
Hydraulic grade line slope ratio (S1/S2) = 1
Ratio of Hazen-Williams formulas for lined and unlined pipe reduces to $\frac{C1}{C2} = \left[\frac{D2}{D1}\right]^{2.63}$

Calculate

$$\frac{C1}{C2} = \frac{90}{110} = 0.82$$

$$\text{For } \frac{C1}{C2} = 0.82; \quad \frac{D2}{D1} = 0.93$$

Required D2 = 0.93 x 200 = 186

$$\text{Allowable lining thickness} = \frac{200 - 186}{2} = 7 \text{ mm}$$

The above example illustrates that cement mortar lining can restore the pipe to its original design capacity or very close to it.

MITIGATION BY CATHODIC PROTECTION

Cathodic protection is an electrical method of preventing corrosion of buried metal objects such as watermains. The principal is to supply sufficient electrical current to overcome the energy loss from the pipe by the corrosion process which consists of chemical reactions and transfers of electrical energy where the metal surface contacts the soil. A galvanic cell is created using the soil as an electrolyte, whereby a sacrifical metal is made the anode (corroding) half of the cell and the pipe is made the cathodic (protected) half of the cell.

The two practical methods of applying cathodic protection are galvanic anodes and impressed current systems. Galvanic anodes consist of magnesium or zinc blocks which are connected to the pipe. Impressed current cathodic protection systems use rectifiers to convert alternating current (AC) electrical power to direct current (DC), which is then applied to a buried anode made of graphite or cast iron.

Two characteristics of the installed pipe which affect cathodic protection are electrical continuity and extent of graphitization. Electrical continuity is dependent on factors such as joint type and use of PVC filler sections at breaks. Graphitization is important because the graphite corrosion product is electrically different from cast iron and may affect the electrical interaction and current distribution between the pipe and the cathodic protection device. Because of these characteristics, cathodic protection is not usually assumed to be 100 percent effective at reducing main breaks and a maximum effectiveness of 80 percent has been used in the subsequent analysis.

Although rectifier-operated impressed current systems would usually be used in high-current applications such as pipe having no protective coating, these systems are not desirable in urban areas because they can accelerate corrosion on other buried utilities such as gas lines.

Consequently, the ideal alternative for complete protection is the distribution of galvanic (sacrificial) anodes along each main to be protected. The density of current discharged by the anodes would be diffuse and would minimize the chances of interference on other utilities and discontinuous parts of the watermains.

ANALYTICAL MODEL AND ECONOMIC ANALYSIS

The model, and subsequent economic analysis, was developed to investigate the alternative break reduction measures as opposed to continued repair of the existing mains.

The logic of the model is shown in Figure 3 which shows that the comparison is between the costs associated with a break reduction alternative and the estimated time profile of break repair savings.

Unit costs based on previous experiences were developed for each alternative and input to a standard present-worth economic analysis. Key assumptions made in the analysis are:

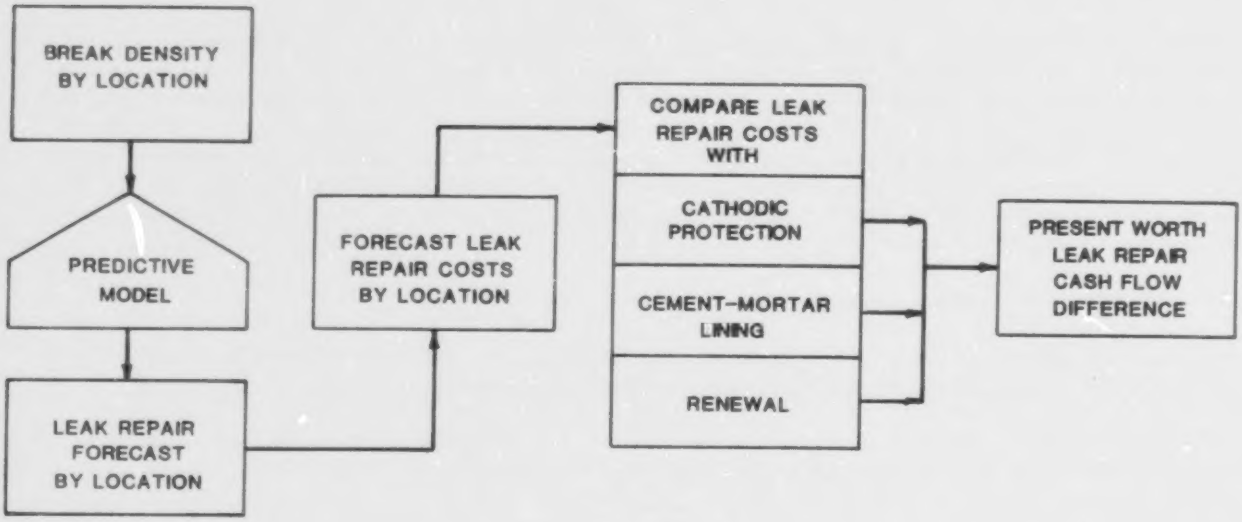
- Effective life of cathodic protection : 15 years with a reduction in break repair frequency of 80% for the first 10 years and 50% for the last 5 years.
- Effective life of cement mortar lining : 20 years with a reduction in break repair frequency of 75% for the first 15 years and 25% for the last 5 years.
- No monetary costs were assigned for interruption of water service and traffic.

Unit costs used in the analysis are:

– Cathodic protection	$100,000/km
– Cement Mortar Lining	$100,000/km
– Renewal	$379,000/km
– Repair	$ 2,513.36 each

FIGURE 3

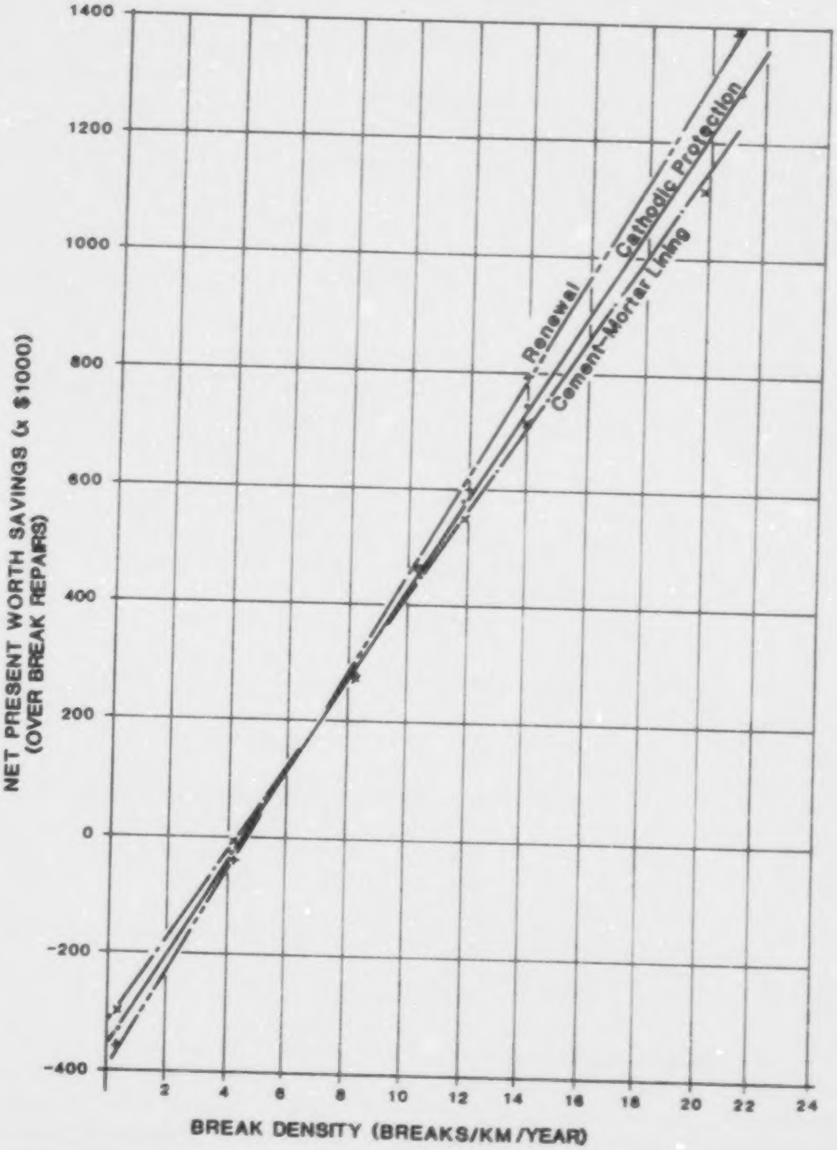
Analysis Model


BREAK DENSITY BY LOCATION
PREDICTIVE MODEL
LEAK REPAIR FORECAST BY LOCATION
FORECAST LEAK REPAIR COSTS BY LOCATION
COMPARE LEAK REPAIR COSTS WITH
CATHODIC PROTECTION
CEMENT-MORTAR LINING
RENEWAL
PRESENT WORTH LEAK REPAIR CASH FLOW DIFFERENCE

The analysis was carried out for 37 selected grids covering a representative range of break densities. The results, as shown in Figure 4, indicated that for higher break densities renewal was most cost effective, while for lower break densities continued repair was preferable. A further present-worth analysis using the same assumptions and unit costs was done on a typical 1 km length of pipe, which could be located anywhere in the city, over a range of break densities.

FIGURE 4

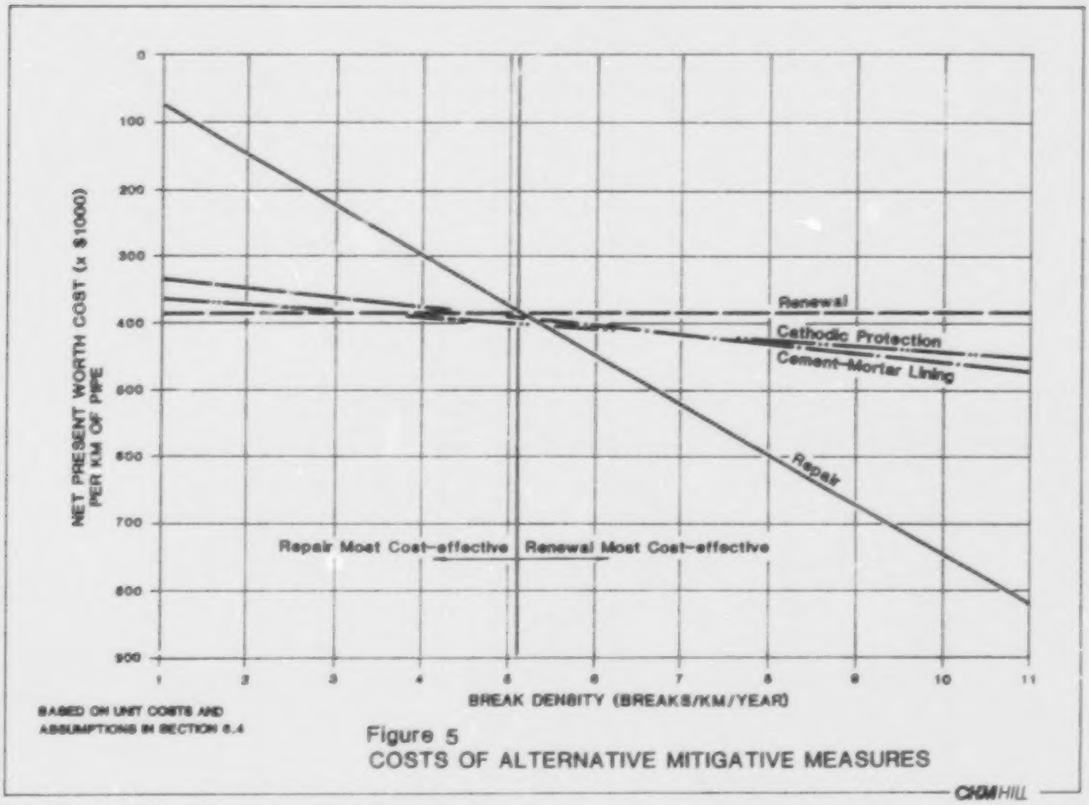
Cost Effectiveness of Break Mitigation



The results, shown graphically in Figure 5, indicated that for watermains with average break densities greater than just over five breaks per km per year, renewal was the most cost effective measure. Cathodic protection and cement mortar lining were also economically preferable to continued break repairs Below about five breaks per km per year however, continued repair was most cost effective.

FIGURE 5

Costs of Alternative Mitigative Measures



To assess the effects of the assumptions made, sensitivity analyses were carried out for changes in inflation rate, for the addition of a service interruption cost, and for changing the life of a protective measure. The results, shown on Table 6, show that increasing any of these variables has the effect of increasing the net present-worth savings of any alternative.

TABLE 6A

ECONOMIC IMPACT SENSITIVITY ANALYSIS
Impact of Changes in Inflation Rate without Changing Discount Rate

Estimated quantity of pipe 1.80 km

Grid 45

Average Break Density 4.9

Inflation Rate	Cathodic Protection	Cement Mortar Lining	Renewal
0.03	$ (175,480)	$ (124,606)	$ (388,743)
0.04	(154,316)	(105 872)	(324,538)
0.05	(115,894)	(72,952)	(237,427)
0.06	(51,230)	(17,314)	(117,278)
0.07	53 910	74,281	51,020
0.08	221,222	222,321	290,148
0.09	483,815	458,451	834,314
0.10	892,270	831,482	1,135,328

Notes:

1) Assume a 10 percent discount rate.

2) Values in brackets are negative indicating that continued break repair is more cost effective than the break reduction alternative

TABLE 6B

Impact of Including Interruption Costs Equal to Repair Costs

	Cathodic Protection	Cement Mortar Lining	Renewal
Without interruption cost	$ 53,810	$ 74,281	$ 51,020
With interruption cost	736,880	718,076	784,241

Note: Assumes a 7 percent inflation rate and a 10 percent discount rate.

TABLE 6C

Impact of Changing Lifetimes of Protection Measure

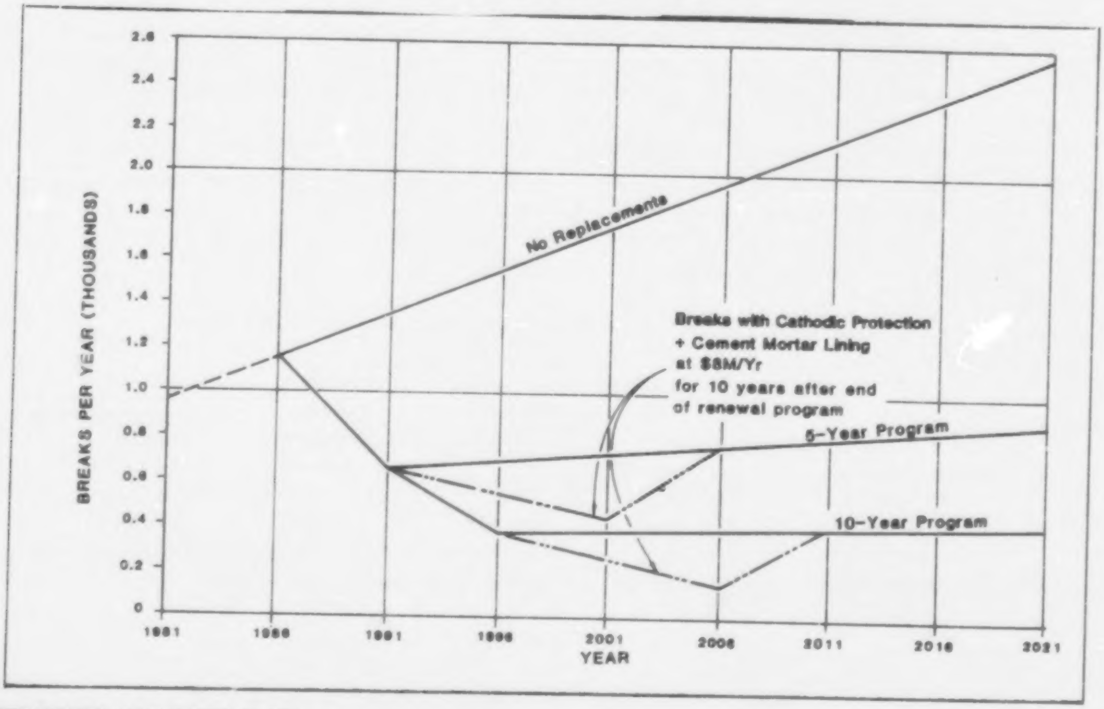
Protection Measure	Lifetime	Net Present Value Saving
Cathodic protection	10	$ 17,570
Cathodic protection	15	53 910
Cement mortar lining	15	64,812
Cement mortar lining	20	74,281
Cement mortar lining	25	78,482
Renewal		51,020

The results of this type of analysis can be used to develop a range of strategies for future activity. Since the overall break rates are expected to decrease the most in early years of the renewal program as the worst pipe is removed from the system, alternative strategies based on this expectation were evaluated to determine their effects on the overall system. Bearing in mind the city's commitment to total system replacement over a 55-year period, the strategies assessed the effects of shorter term renewal programs and also the effects of subsequent mitigative measures.

As expected, the results, presented on Figure 6, indicate a rapid decline in the number of breaks in the initial 5- or 10-year renewal programs. If no further work is carried out on the system breaks gradually begin to increase again. If mitigative measures are applied, a continued reduction in breaks is anticipated until the mitigation is stopped. A similar analysis could be carried out to evaluate any number of alternative strategies.

FIGURE 6

Effects of Mitigative Strategies



CONCLUSIONS AND FUTURE PLANS

The work described in this paper[1] has led to a number of conclusions about Edmonton's cast iron watermain system and the city's future plans for dealing with the watermain break problem.

1. The present renewal identification system is effectively removing pipe with the highest break densities from the distribution network and should be continued

2. Renewal is the most cost-effective way of mitigating breaks for mains having average break densities greater than about five breaks per km per year.

3. Continued repair is the most cost-effective way of treating the break problem at average break densities less than five breaks per km per year.
4. The city has initiated a Cathodic Protection Test Installation designed by CH2M HILL to evaluate that system as a mitigative measure.
5. The city is investigating installing a trial section cement mortar lining, both as a "C" factor restoration measure and as a corrosion mitigative measure.
6. The city has asked CH2M HILL to produce a corrosion protection manual and also to upgrade the city's construction specifications for corrosion protection measures such as coatings, anode installation, and electrical continuity.
7. The city intends to continue to update the data base on break frequency and location so that the analysis of break projections and mitigative strategies can be updated as additional information becomes available

Corrosion studies as discussed in this paper can be used for:

- establishing budget priorities
- predicting future funding needs, and
- economic assessment of alternatives

and can be a valuable tool in water system planning.

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BACTERIOLOGICAL ASPECTS OF IRON AND MANGANESE IN WATER

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The presence of iron and manganese in water has long created problems in the delivery of an adequate water supply for domestic, agricultural and industrial uses. The problems created by iron and manganese are multiple. They relate to the impact that the iron (and manganese) has on the color of the water, turbidity, taste and odor and the reliability of the supply itself. In the past, there has been a tendency to look upon the iron problems occurring in water as having a purely chemical nature involving a series of oxidation-reduction reactions with the subsequent production of sesquioxides, oxides and hydroxides which, upon deposition, eventually led to reductions in flow due to the plugging of pipes and equipment. It is only in the last two decades that the role of microorganisms in the cycling of iron in water has been addressed in more detail although the original linkages between bacteria and the iron cycle were established in the late 19th century principally by Winogradsky.

The effects that iron can have on water are multiple and interrelated in a series of complex patterns with the bacterial flora dynamically present in the various stages of the water treatment to usage process.

Primarily, it is necessary to define more precisely the roles that the iron (and manganese) can play in causing direct problems. In terms of the coloration of the water, this may be as a result of the presence of iron in either the oxidized insoluble ferric form in which case the color is due to particles of iron containing material being present in the water; or the reduced ferrous soluble form whereby the color would be due to the dissolved ferrous salts present in the water Where the ferric irons are present, the iron will tend to be in particulate form, whereas in the ferrous type, the iron will tend to be dissolved in the water. The color of the water will vary with the particular form of iron. Where the iron is in an oxidized form, the water will tend to be "rusty" in color having an orange to brown hue and be relatively turbid. When the ferrous iron is present, the water will tend to be yellower in color due to the presence of the ferrous iron.

The same color patterns do not occur with manganese. In general, where a water contains a high manganese content and tends to be of a much darker brown or even black It must be recognized that many waters contain mixtures of iron and manganese which will also cause a variation to the basic colors described above. The occurrence of hydrogen sulfide as a result of anaerobic decomposition (with corrosion) will further darken the colors because of the presence of the black metallic sulfides.

Cloudiness in a water particularly when it is accompanied with a rusty color automatically indicates the potential for an iron problem in the water. Where a water is turbid this indicates that there is a sufficiently high concentration of particulate material so that the eye is able to pick up these interferences to the passage of light. Often a deposit will also develop in the water as the larger particles settle out. Rusty deposit collecting at the bottom of a water sample, if it occurs rapidly, would indicate that the ferric salts were present in the original water sample. However, if the precipitate collects over the course of a day or two, then this could indicate that the iron was in the ferrous form with only a low concentration of ferric dissolved salts. Gradual oxidation of the ferrous salts as the water sample was left standing (and particularly accelerated if chlorine has been added to the water) to the ferric form which would precipitate rapidly as this oxidation occurred. There are many other causes for turbidity in water including the suspension of insoluble chemical salts such as carbonates, the presence of organic particles and microbial cells. The presence of the iron can be easily proved by the oxidation of the water sample wherein any ferrous and manganous salts will oxidize and precipitate out to give a colored deposit. Diagnosing the other particles in the water becomes more difficult. From a surface-water supply, high particulate loadings may be developed as a result of algal blooms, sediment disturbance and general biological growth. In water coming from a ground water system, such a turbidity may be generated by various chemicals reactions involving oxidation, changes in pH and the sporadic releases of bacteria from the biofilm which may be forming within the aquifer, the well or the equipment used to pump and treat the well water.

There is a growing need to develop the technologies for determining the size of the particles in order to diagnose more completely their origin. For sample, an optically clear water with a zero turbidity could contain up to 100,000 bacteria per ml. The fastest way to determine the presence of such a high number of bacteria without resorting to laboratory examination by the standard or total plate count methods would be to count the number of particles using the lazer particle counter. Such a survey could be completed in the laboratory in 15 minutes and would reveal the number of particles in the .5 to 5 micron range (if the cells were dispersed in a planktonic form) or in ranges of particles up to 125 microns in diameter (if the bacteria are dispersed in a particulate form). If the bacteria are dispersed in a particulate form, it is probable that the origin was a biofouling situation within either the ground water system, the water treatment plant and/or the surface water. By examining the changes in the particulate spectrum in terms of the volume of particles at different sizes and also the numbers, it is possible to more accurately diagnose the relevance of the turbidity to an iron fouling problem and also project the likelihood of the biological component being a significant cause of the perceived problems.

Taste and odor can arise from iron and manganese directly since higher concentrations will give the water a somewhat metallic taste. Odor is, however, not normally derived directly from iron and manganese but can become associated where there is a bacteriological factor involved. This odor may take a number of forms depending upon the nutrient and oxygen status within the system. Low oxygen concentrations in a water will encourage the anaerobic decomposition of organic materials. One of the major products of this type of decomposition is hydrogen sulfide which will give primarily a "bad egg" smell to the water and secondarily, it will react with any iron and manganese (along with other metals) to form black sulfide deposits. The presence of hydrogen sulfide and the bad egg smell would also indicate that there is a high risk of corrosion occurring within the system. Most of the hydrogen sulfide is produced either in slimy growths (biofilms) occurring within the sediment, aquifer, wells or treatment systems and the odor will be detected only if it has not been totally neutralized into blacken sulfide deposits. Other odors which can originate from bacteria include earthy musty (due to the aerobic growth of Streptomyces), fishy smells (often associated with the pseudomonas bacteria) and fecal smells (often generated by members of the coliform bacteria group). While bacteria in water do create some taste problems, there has been less attention paid to the precise flavors that are generated by these organisms and most of the attention is paid to diagnostic procedures for odors.

The reliability of a given water system is influenced by the amount of iron and manganese in the water in a number of ways. These can be categorized into two basic groups. Firstly, the production of the oxidized ferric salts either through chemical or biological action will lead to their deposition either as a salt deposit or in a slimy biofilm within the water system which will impair productivity through reducing channel sizes and creating resistance to flow. Secondly, the presence of iron and manganese in the water may also stimulate corrosive processes through providing (upon deposition) a larger environment within which anaerobic SRB (sulfur reducing bacteria) are able to grow and produce corrosive hydrogen sulfide. While initially the function of the iron would be to neutralize the hydrogen sulfide by reaction to metallic sulfides. Concurrently, hydrogen sulfide will also set up an electro-chemical corrosive environment within the plant.

The bacteria involved in dynamic changes to iron and manganese in water traditionally have been called the iron bacteria because they appear to be associated totally with brown slimy growths on surfaces associated with the "infected" water. Of these iron bacteria, the dominant organism has generally been considered to be *Gallionella*. This bacterium produces a very long tail from the side of the cell which is ribbon-like and laced with iron and manganese deposits to give it a characteristic orange to brown color. More recent research has suggested that the iron bacteria could, in fact, be divided into three major groups based upon their ability to utilize the energy released by the oxidation of the iron and manganese, their ability to produce an extensive cellular coating (polymer) within or upon which the cell grows and thirdly, the cbility of the organisms to utilize organic carbon. There is a tendency now to refer to these iron bacteria as iron related bacteria (IRB) because all three groups have a common ability to concentrate iron and/or manganese oxides and hydroxides either within the polymer, the cell wall or within the cell proper. All of the IRBs require oxygen for growth at the time of this assimilation. However, the three groups are very different in their basic habitat requirements.

Group 1 contains the genus *Gallionella* which is able to utilize energy from the oxidation of iron, deposit the oxides and hydroxides within a very defined ribbon-like polymeric structure excreted from one side of the cell, and has a low requirement for both oxygen and organic material. Consequently, *Gallionella* is often dominant in waters that are low in organic carbon as well as oxygen.

The second types of iron bacteria also produce polymeric extensions from the cell but usually in the form of tube-like processes within which one or more cells may sit. In some cases, cells are able to abandon these tubes of polymers and move elsewhere. While it is thought that some of this Group 2 may be able to use some energy from the oxidation of iron and manganese, most of the energy is obtained by the degradation of organic compounds. There are many genera involved in this group and all are generally associated with the generation of thick iron bacterial slimes occurring at the oxidation-reduction interface whether in an aquifer, on a well screen, pump filter or in a treatment process.

Group 3 consists of a much larger group of bacteria including some of the coliforms. The basic differentiation of this group is that as a part of the normal processes of growth, the bacteria take up iron and manganese oxides and hydroxides which are deposited in the polymers which may be excreted around individual cells or in the cell walls themselves. Rarely, it appears, are the oxides and hydroxides laid down with the cell cytoplasm itself. These bacteria appear to form, from present research studies, approximately one to ten percent of the normal bacterial flora found within a water system. When iron is present, these bacteria will take up the oxides and hydroxides to produce a brown slimy growth typical of the classifical iron bacteria group. These bacteria often grow as a group and are able to survive in a mutualistic consortium.

The Group 3 IRBs are particularly important because they frequently dominate in terms of the cell numbers within a slime and also readily respond to changes in environmental conditions. For example, a reduction in the oxygen loading will cause some of the bacteria to generate hydrogen sulfide production causing the slime to turn a black color. Additionally, if iron and manganese no longer appear in the water supply or is not present, the slime can form based upon the organic nutrient feed stock to produce a white or grey slime. An iron or manganese deposit which contains an element of biological activity may be estimated by simply rubbing some of the slime material between the finger and the thumb. If the deposit is very granular and has no "slippiness", then the probability is that the deposit is fundamentally chemical in nature. However, if there is a slimy feel to the material, in all probability, there is an IRB growth and the polymers that have been produced are actually creating the sliminess. Perhaps a prime example of an excessive slime growth is on the Titanic which has some very dramatic stalactitic growths of iron bacterial which have generated over the last 75 years.

The types of habitat that the IRBs can be located varies from group to group. The most specific habitat is for Group 1 which would normally be found occurring in a low organic water where the oxygen levels are suppressed Often the Group 1 Gallionella IRBs are recognized very rapidly by microscopic examination and frequently their role in a water system may be over-emphasized simply because the bulk of the material seen in the water are the ribbon-like polymeric processes produced by Gallionella. However, the cell of this organism is relatively small and sits on one end of the stalk. At one point in the life cycle, this cell can detach from the stalk and migrate away to a more suitable environment where a fresh stalk may then develop. The migrating cells resembles a pseudomonas and there is some possibility that a number of pseudomonads can, in fact, generate a Gallionella like stalk under specific environmental conditions.

The Group 2 IRBs have a broader habitat and are frequently found in waters with a higher organic loading and oxygen. The third group appears to function in a broader range of habitats still where there is the passage of more adequate organic loading but is not necessarily dependent upon the presence of oxygen since nitrate can be substituted by many of the organisms for oxygen to achieve respiration.

The way that the IRBs function in Groups 2 and 3 is to develop as a biofilm in which the cells sit within a polymeric mass of slime. This slimy biofilm remains in contact with the water so that any nutrients passing through the water that come into ultimate contact with the slime are rapidly taken up by the biofilm Studies on the retention of these nutrients have shown that there is a high degree of reutilization of the nutrient within the slime so that even if there is a low nutrient supply in the water, the constant supply of nutrients within the slime can lead to very healthy ongoing growths.

Slimes undergo a number of phases of growth starting with bacterial attachment to the surface, limited colonization over the surface, development of microcolonies (clusters of reproducing cells), generation of a large polymeric "umbrella" which will protect the organisms to some extent from adverse environmental conditions (e.g. chlorination), coalescence of the slime into a large rapidly growing mass of cells within an expanding slime, sloughing of the outer edges causing polymeric finger-like processes to be extended into the water causing an increase in friction (and flow rate loss), and finally the fusing of the various surface slimes into a common mass to completely restrict flow and cause a terminal plugging. The length of time required to complete this process is affected by temperature, oxygen levels, pH, nutrient availability and rate of flow. While it may be thought that the faster the rate of flow, the slower the extension of the biofilm will be, it has generally been found that the higher flow rates simply produces a much more densely packed slime mass which still has the capability of extending and eventually causing plugging.

Plugging may be viewed in two stages. The first stage involves a restriction of the functional space through which the water can pass unimpeded while the second phase would occur when there is a total restriction of flow caused by the total displacement of free flowing space at one continuous point within the system.

The basic scenario exists where a biological plug can occur. These are as a biological coating on the surface matrix over which the water flows through the interstitial spaces past the slots in the screen Further restriction to flow can occur here as a result of the growth of the biofilm across the slot which may eventually plug the water flow at the well screen or pump filter. Biofilms can also occur within the pipe to reduce the effective internal diameter through which the water can flow. The first symptoms of such a plugging in a water system could include increased pumping pressures, reduced flow, increased draw down, increased turbidity and sometimes radical shifts in the product s water quality parameters [particularly turbidity, coliforms, total plate count and nutrients].

Traditional chlorination/disinfection practices are frequently not able to control the plugging phenomenon completely or for a long period of time because of the nature of the polymeric "umbrella". Here when the chlorine comes into contact with the polymers, it is neutralized and therefore unable to penetrate deeper into the biofilm unless excessively high dosages are used with prolonged exposure times. In addition, the bacteria composing the slime consortium have to be sufficiently chlorine sensitive to cause death and dispersement.

Different scenarios have been projected for the control of plugging ranging through from prevention to restoration In the prevention mode, serious obstacles arise because of the universality of the bacteria within aquifers, wells and water treatment systems except at points where high levels of disinfection are constantly being practised. At this time, the main mechanisms of prevention relate to moving the biofouling plug to a site where it will cause less problems to the water system. For example, where plugging has been occurring in the gravel pack and well screen zone, this can be moved into the aquifer by shifting the oxidation-reduction fringe outwards into the ground water system by the injection of aerated water either directly down the well column or through a series of injection wells set peripherally around the producing well This latter process is known as the Vyredox (TM) process. Where the position of plugging is moved out into the ground water system, a much larger surface area is presented over which the biofilm can grow so that the life expectancy of the producing well can be extended considerably before plugging can occur. An additional advantage is that the process of iron and manganese removal is performed by the IRB plug at the new site so that 80 to 95% of the iron and manganese can be removed and retained by the biofilm to give a much better quality product water flowing into the well.

A second promising method for the prevention of plugging in the well is by the application of low dosage gamma irradiation. In East Germany, a large number of wells are being maintained by the use of in situ irradiation devices installed at the screen level down hole. Various other methods are presently being tested to determine the most satisfactory safe way to utilize gamma irradiation for the restriction of plugging in water wells

Where a well has already been subjected to severe plugging with a very significant loss in flow and increase in draw down to the point perhaps of total plugging, then a process of restoration has to be applied to the well. The traditional practices of "shock" chlorination and acidization can work effectively in the short term where the biofilm happens to be composed of microorganisms sensitive to pH shifts and/or chlorine to the point that dispersion of the biofilm occurs. However, where this has happened there is a very strong probability of a rapid recurrence of the plug once treatment has been completed.

Another restoration scenario which was developed in Saskatchewan in the 1970's is the application of heat (pasteurization). The basic premise for this method is that microorganisms able to resist disinfection practices by virtue of the polymeric "umbrella" would remain very sensitive to heat which would easily penetrate through the protective matrix and kill the cells.

From 1975 to 1982, various methods of well pasteurization were developed including direct steam injection, hot water recycling, electrical immersion heater implantation, microwave irradiation and steam heat tube injection. A number of plugged wells were brought back onto stream by this technology and maintained for various periods of time. However, there was some concern that pasteurization does not function so efficiently on a combination of chemical precipitation/saturation phenomena along with the biological plugging. In order to study this, a series of model wells were developed in 1985 to achieve a plugging with IRBs in relatively short periods of time (as little as 100 days). These plugged wells were then subjected to various treatments. As a result, a blended treatment method has been developed which is now subject to patent application. This method involves a combination of prechlorination (in order to destroy the surface biofilm) coupled to the sequential use of a very effective surfactant and heat to disperse the damaged biofilm. The final phase involves a chemical post treatment and surging in order to ensure the maximum removal of the plug from the well itself. Trials are presently underway in the southern United States to determine the effectiveness of this treatment for the restoration of both relief and water wells.

The phenomenon of iron bacteria being functionally able to remove the iron and manganese by oxidation from the water into the biofilm also has a considerable potential as a treatment system. Stimulating the biofilm to precipitate and/or remove the iron and manganese can be achieved by the provision of a large enough surface area and/or the application of additional oxygen to the system. A number of approaches exist including one already referenced (Vyredox system). In Finland, iron removal is being widely achieved (85% or better) by the free flowing of the iron laden water over a rock/coarse gravel bed where the water is exposed to high levels of air as the then surface of films flow over the stones. Once the water has passed over this zone, it collects in an entrapment zone and on into the water reservoir. Considerable success has been developed using this technique for smaller communities in the country. The problem with applying that technology in the prairie would be excessively cold winters.

Aeration devices where water droplets sprayed through a circulating air current has also been found to rapidly cause the precipitation of the iron and manganese as a brown sludge. This sludge is high in IRBs and the role of biological component in causing the oxidation of the ferrous and manganous salts in the water remains to be elucidated. However, the increased efficiency of these units with time would suggest that an established biological component is essential to the efficient functioning of the unit.

From the evidence being developed in the research laboratories, the iron related bacterial group functions not only in negative ways through the development of plugging and forming of the support matrix for corrosion, but also in a positive way through removing not only iron and manganese from the water but also some of the nutrients. The actual impact of a plugging situation on nutrient removal has not yet been addressed effectively at the research level. It is clear that the biofilm operates as an extremely effective "sponge" absorbing nutrients from the water and retaining them within the slimy polymeric matrix of the biofilm. One of the major difficulties in determining this retention factor has been the inability to obtain water samples of water entering the zone of plugging where these biological activities will occur. In general, well water analysis are all performed on the water taken from the well, distribution system or product water. Such waters have already been subjected to considerable biological activity once the biofilm has formed and are essentially "post diluvian" water, that is to say, water after the effect of the biological activity.

In the next decade, the exact role of this biological zone may become more fully understood and it could be that its role as a primary filtration system for groundwater may be more critical than presently believed and, when stimulated, more efficient.

FOR FURTHER INFORMATION

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RADIOACTIVITY SURVEY OF SELECTED DOMESTIC GROUNDWATER SUPPLIES IN MANITOBA

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BACKGROUND

Initial work was carried out by Manitoba Environment and Workplace Safety and Health in the southeastern area of Manitoba based on the results of a 1978 mineral exploration survey undertaken in that area. In 1978 work suggested that significant levels of uranium would be found in certain groundwater wells based on the presence of a favourable geologic environment. Chemical uranium levels in several wells were found to exceed the National Guideline of 20 ug/L as recommended in "Guidelines for Canadian Drinking Water Quality 1978"[1]. Figure 1 shows the specific areas first examined Beausejour and Thalberg.

The scope of the work was expanded into the general cross-hatched area shown in Figure 1. In November and December of 1982, a "mini survey" of 34 selected groundwater wells and 15 surface locations was carried out by Departmental staff. Five wells sampled had uranium results in excess of the National Guideline with two of the five being well beyond this level.

Additional sampling for chemical uranium was carried out during 1983. Twelve groundwater wells in the designated area were sampled for radioactivity analysis (gross α activity and gross β activity). The analyses were performed by the Radiation Protection Bureau of Health and Welfare Canada in Ottawa. Four of the wells sampled had uranium in excess of 20 μg/L while one of the wells was found to have uranium in excess of 100 μg/L (Interim Manitoba Guideline Level). Four of the wells sampled were reported to have gross α values in excess of the target level[1] of 0.1 Bq/L, while three of the wells were found to have gross β values in excess of the target level[1] of 1.0 Bq/L.

The wells found to be in excess of the gross α and gross β target levels were resampled for radium - 226 analysis and lead - 210 analysis respectively during December of 1983. The specific radionuclide results were found to be below the recommended target levels[1].

The specific work being carried out in the southeastern area of Manitoba was eventually integrated with the background survey work being carried out in the area of the Underground Research Laboratory being established by the Whiteshell Nuclear Research Establishment near Pinawa.

1980 RADIOACTIVITY SURVEY

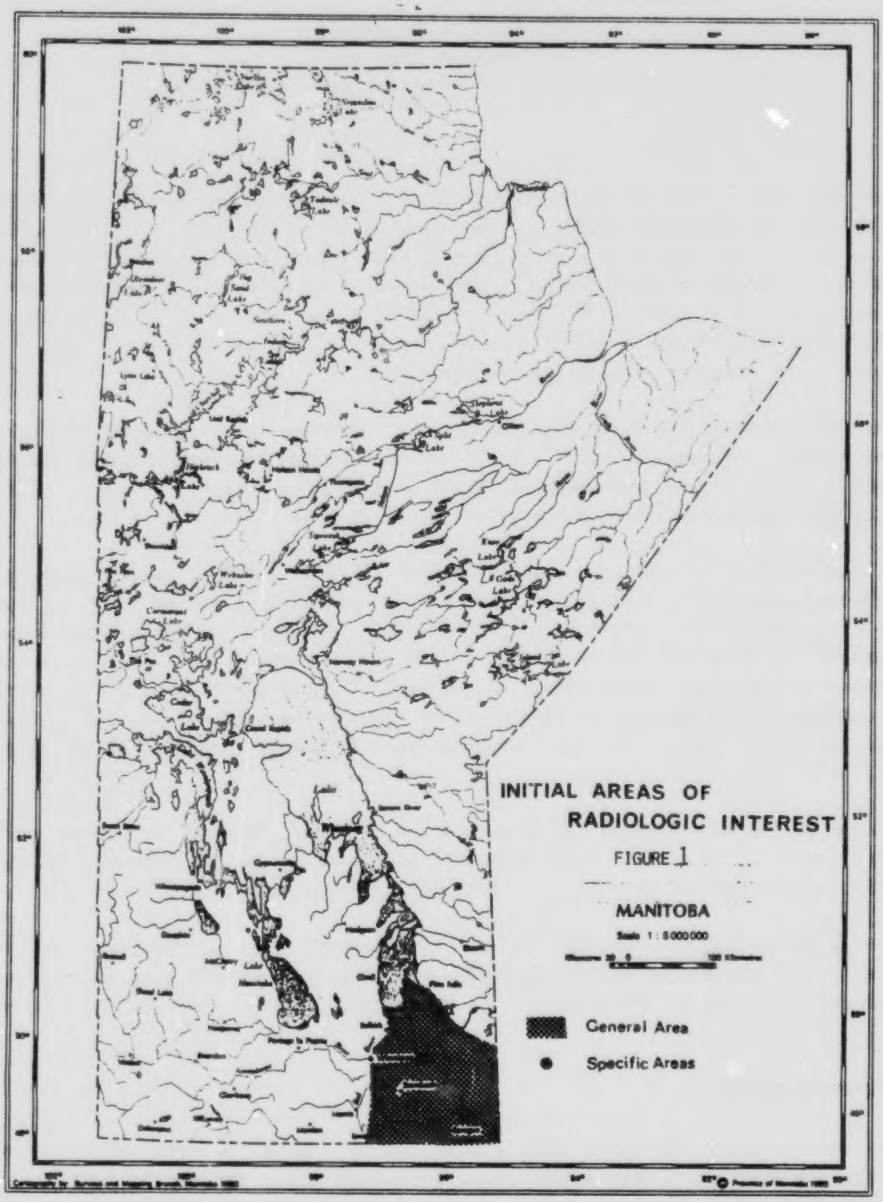
Manitoba Environment and Workplace Safety and Health has as a part of its mandate the responsibility for monitoring and surveillance of public water systems in Manitoba. In this regard, Departmental staff routinely sample such systems for chemical analysis.

In 1980, it was decided to proceed with a mini radiologic survey on selected public water systems. This decision was based on the fact that:

1. radiologic monitoring of public water systems is recommended and such monitoring had not been done in Manitoba; and

FIGURE 1

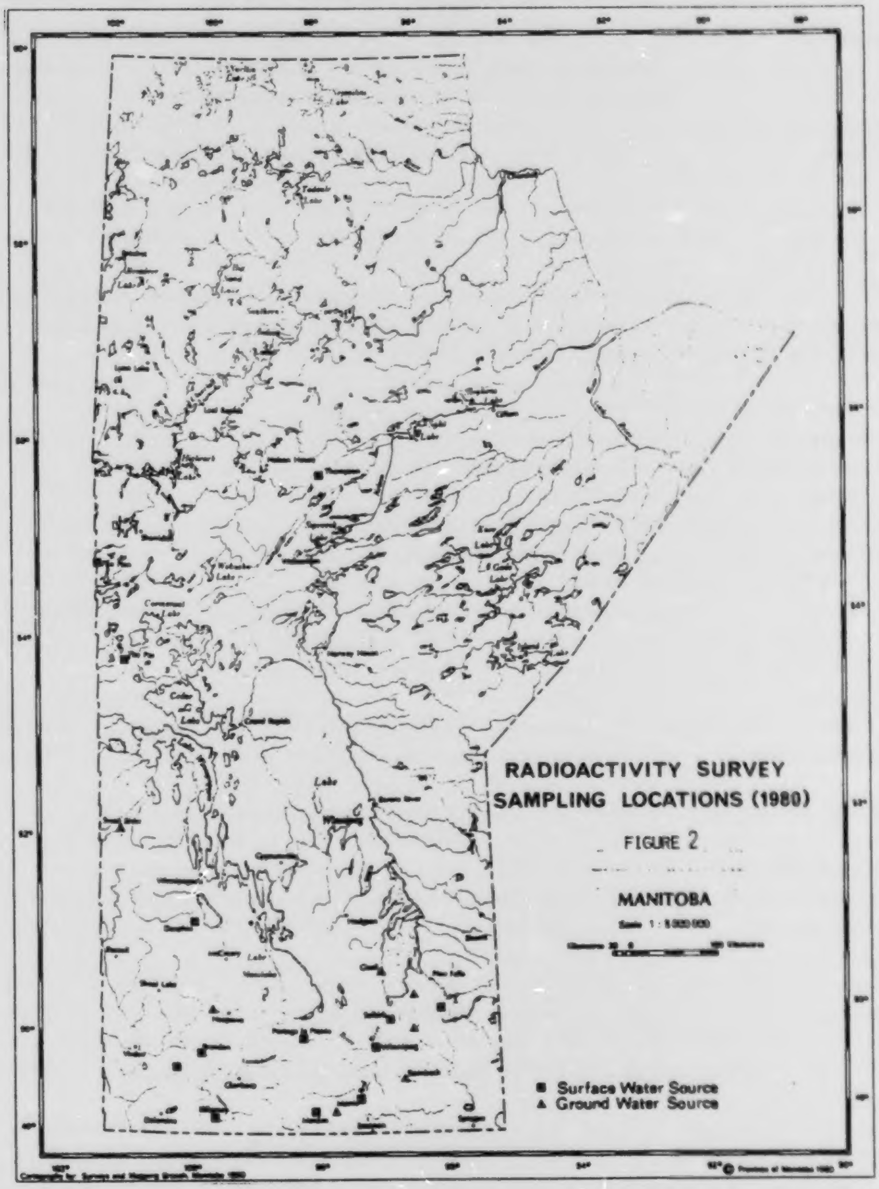
Initial Areas of Radiologic Interest


INITIAL AREAS OF
RADIOLOGIC INTEREST
FIGURE 1
MANITOBA
Scale 1 : 5000000
General Area
Specific Areas

2. elevated chemical uranium levels had been found in some groundwater wells in southeastern Manitoba.

The survey involved 20 sampling locations in Manitoba as shown in Figure 2. The 20 locations included 19 public water systems and 1 private groundwater system. Both surface water supplies (13 locations) and groundwater supplies (7 locations) were included in the survey, which concentrated on the large population centres in Manitoba.

FIGURE 2
Radioactivity Survey Sampling Locations


RADIOACTIVITY SURVEY
SAMPLING LOCATIONS (1980)
FIGURE 2
MANITOBA
Scale 1 : 5 000 000
■ Surface Water Source
▲ Ground Water Source

Analytical parameters for the 1980 Survey included gross α activity, gross β activity, strontium - 89, cesium - 137 and tritium. Three of the stations were also sampled for strontium - 90. The radiologic analytical work was carried out by the Radiation Protection Bureau.

Table 1 provides the sampling locations by name and the radiologic results for the 1980 survey. The results were considered to be normal and were within guideline values recommended for drinking water. Table 2 provides the recommended limits for radionuclides in drinking water that correspond with the parameters of the 1980 survey. All values were extracted from "Guidelines for Canadian Drinking Water Quality 1978"[1] except the value for strontium - 89[2].

TABLE 1

Results of 1980 Radioactivity Survey of Selected Public Water Systems in Manitoba

Location	Raw Source	Gross Beta Bq/L	Strontium -90 (Bq/L)
Thompson	Surface	0.12 +/- .02	0.03 error .003
The Pas	Surface	0.10 +/- .02	ND
Flin Flon	Surface	0.13 +/- .02	ND
Winnipeg	Surface	0.18 +/- .02	0.09 error .007
Brandon	Surface	0.26 +/- .03	0.01 error .001
Selkirk	Surface	0.08 +/- .06	ND
Portage la Prairie	Surface	0.23 +/- .04	ND
Steinbach	Ground	0.14 +/- .06	ND
Beausejour	Ground	0.23 +/- .07	ND
Lac du Bonnet	Surface	0.06 +/- .05	ND
Morden	Surface	0.40 +/- .15	ND
Dauphin	Surface	0.12 +/- .05	ND
Gimli	Ground	0.34 +/- .08	ND
Swan River	Ground	0.25 +/- .06	ND
Souris	Surface	0.25 +/- .09	ND
Killarney	Surface	0.50 +/- .06	ND
Minnedosa	Ground	0.34 +/- .10	ND
Winkler	Ground	0.07 +/- .02	ND
Morris	Surface	0.15 +/- .09	ND
Private Well	Ground	0.35 +/- .10	ND

Values for the following parameters for all the above locations were as follows:

Gross Alpha	Lt 0.05 Bq/L
Strontium -89	Lt 0.001 Bq/L
Cesium -137	Lt 0.001 Bq/L
Tritium	Lt 20 Bq/l

TABLE 2

Recommended Limits for Radionuclides in Drinking Water

Radionuclide	Maximum Acceptable Concentration Bq/L	Target Concentration Bq/L
Cesium-137	50	5
Radium-226 (Gross)	1	0.1
Strontium-90 (Gross)	10	1
Strontium-89	300	30
Tritium	40,000	4,000

Recommendations that arose from the 1980 Radioactivity Survey were as follows.

1. The radiologic results with an interpretation should be provided to the system owners.

2. A larger radiologic survey of public water systems should be conducted in Manitoba in three to five years.

The radiologic results were provided to the system owners and it was decided to conduct a large scale survey of public water systems in 1984.

1984 RADIOACTIVITY SURVEY

The 1984 Radioactivity Survey was designed to include 230 groundwater stations. Surface water stations were not included because elevated naturally occurring radiologic contaminants would not be expected to be in Manitoba surface waters. This assumption was confirmed by the results of the 1980 survey.

The 130 stations included in the survey involved all known groundwater public water systems with distribution networks (87 locations), selected communities having public standpipes served by groundwater (15 locations) and selected larger communities relying on private wells (28 locations). Figure 3 provides a picture of the scope of this 1984 Radioactivity Survey. The survey was considered to be fairly representative of groundwater conditions in Manitoba.

Raw water samples were taken at the sampling locations where possible. The raw water was sampled to allow for quality extrapolation to other wells in the area drawing from the same aquifer.

Well logs were obtained for the locations sampled where possible. The logs provided information on the groundwater aquifer being sampled and, once again, could be utilized for extrapolation purposes.

Radiologic parameters included in the 1984 Radioactivity Survey were chemical uranium (to be followed by isotopic analysis if necessary), lead - 210, radon - 222, radium - 226 and radium-228. The radiologic analyses were once again performed by the Radiation Protection Bureau. The radiologic parameters are listed in Table 3.

Samples were also analyzed for a general chemical and heavy metal screen at the W.M. Ward Technical Services Laboratory. The parameters are listed in Table 3 The intention of doing the chemistry work was to determine if there was any link between chemical and radiological water quality.

The field portion of the 1984 survey commenced during the week of September 24, 1984 and was essentially completed as of November 14, 1984. During most of the survey, two one-man sampling parties conducted the sample collection. A heavy degree of logistics was required to collect, store and ship samples. As an example, a special courier service had to be established to deliver radon samples to Ottawa within 24 hours of collection.

FIGURE 3

Radioactivity Survey 1984 Groundwater Sampling Locations

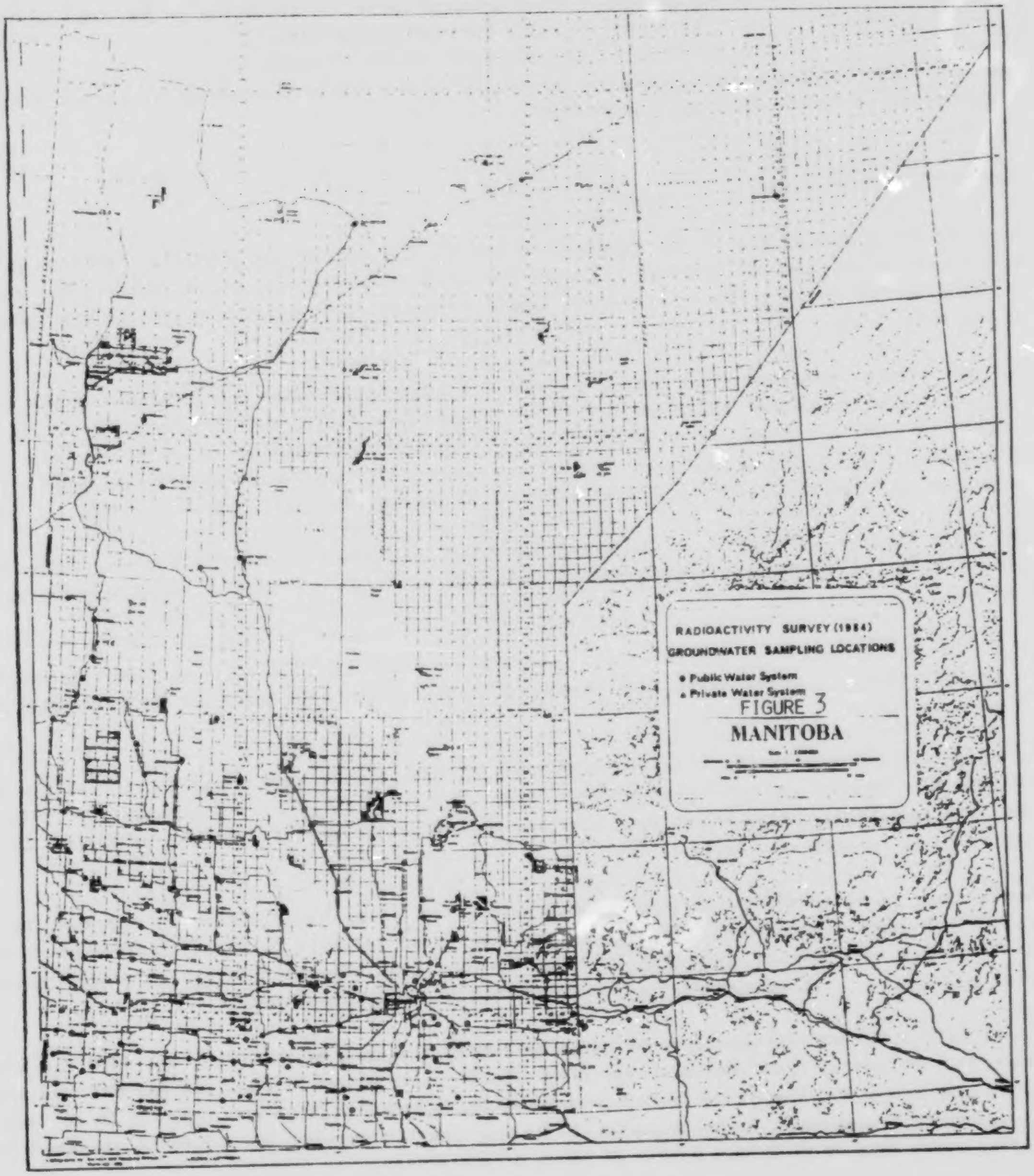

RADIOACTIVITY SURVEY (1984)
GROUNDWATER SAMPLING LOCATIONS
● Public Water System
▲ Private Water System
FIGURE 3
MANITOBA

TABLE 3

Parameters Analyzed for the 1984 Radioactivity Survey

RADIOLOGIC PARAMETERS	CHEMICAL PARAMETERS	
Uranium (Chemical Screen)	Conductivity	Filterable Residue
Lead - 210	pH	Alkalinity
Radon - 222	Calcium	Magnesium
Radium - 226	Hardness	Manganese
Radium - 228	Iron	Sodium
	Potassium	Fluoride
	Nitrate-Nitrite	Chloride
	Sulphate	Arsenic
	Cadmium	Copper
	Lead	Zinc
	Boron	

RESULTS AND DISCUSSION - 1984 RADIOACTIVITY SURVEY

As was previously indicated, the sample collection was essentially completed as of November 14, 1984. Most of the well logs have been obtained, the chemical analyses have been completed and the radiologic analyses have been completed except for radium - 228.

Table 4 provides a summary of radiologic results for the 1984 Radioactivity Survey. Minimum, maximum, mean and guideline values are given for the various parameters. The mean values were calculated including the numeric values for the "less than" results. The uranium guideline value is the Manitoba Interim Guideline, while the values for lead - 210 and radium - 226 are based on recommendations by the International Commission on Radiological Protection[3].

TABLE 4

Summary of Results for 1984 Radioactivity Survey of Selected Groundwater Supplies in Manitoba

PARAMETER	MINIMUM VALUE	MAXIMUM VALUE	MEAN VALUE	GUIDELINE VALUE
Uranium	LT 0.1 ug/L	95.8 ug/L	8.9 ug/L	100 uh/L
Lead-210	LT 15.0 uBq/L	75.8 mBq/L	19.4 mBq/L	300 mBq/L
Radon-222	LT 0.3 Bq/L	36.1 Bq/L	10.1 Bq/L	N.A.
Radium-226	LT 5.0 mBq/L	167.8 mBq/L	16.3 mBq/L	1000 mBq/L
Radium-228		RESULTS NOT AVAILABLE		

Specific chemical and heavy metal results for the locations sampled are not given in this paper. The results were found to be generally acceptable with guideline exceedances being reported for nitrate (four locations), fluoride (two locations), arsenic (one location) and lead (one location). Locations with chemical or heavy metal guidelines exceedances were resampled and followup action was taken where necessary.

As is indicated in Table 4, all the radiologic results reported to date from the 1984 survey were within the guideline values used. Fifteen of the stations sampled, however, were found to have chemical uranium values greater than or equal to the present National Guideline[1] of 20 ug/L. Although none of the uranium values reported exceeded the Manitoba Interim Guideline of 100 ug/L, it was decided that it would be prudent to resample all stations that were found to equal or exceed the National Guideline value.

1986 RADIOACTIVITY SURVEY

In September and October of 1986, additional samples were taken by Departmental staff for radioactivity analysis. Radiologic parameters were as per the 1984 survey (Table 3) and the analytical service was provided by the Radiation Protection Bureau.

Eleven of the fifteen groundwater stations that had uranium levels equal to or in excess of the National Guideline were resampled (one well was not available for resampling and three communities had changed raw water sources). Twelve new stations, including the three communities with new groundwater sources, were also sampled.

Figure 4 shows the locations that were sampled in the 1986 Radioactivity Survey. As may be noted from the scatter of the points, the 1984 survey did not highlight a specific area of concern.

The radiologic results from the 1986 survey are not yet available. Once the data is available it will be analyzed and the results will be provided with an interpretation to the well owners.

FUTURE RADIOLOGIC WORK ON GROUNDWATER SUPPLIES IN MANITOBA

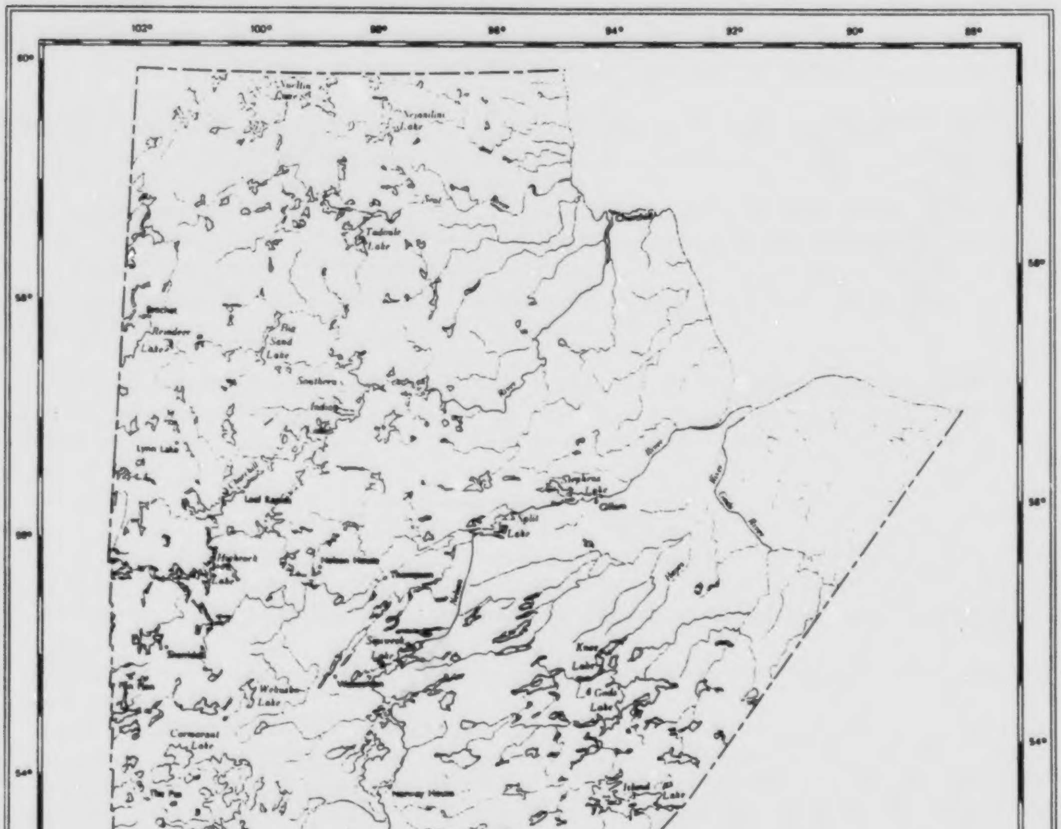
Based on the work carried out in Manitoba, as reported herein, it would appear that, for the most part, concern respecting the potential for naturally occurring radiologic contamination of groundwater supplies may be limited to chemical uranium. Guidelines based on the chemical toxicity of uranium have generally been found to be more restrictive than those based on its radiotoxicity[4].

Recommendations for future endeavours in this area include the following:

1. Necessary followup to the 1986 Radioactivity Survey should be carried out once the results are available.
2. Groundwater drinking water supplies should be routinely examined for chemical uranium.
3. Consideration should be given to carrying out further large scale radiologic surveys of Manitoba groundwater supplies in the future.

FIGURE 4

Radioactivity Survey Resample Locations


RADIOACTIVITY SURVEY
RESAMPLE LOCATIONS (1986)
FIGURE 4
MANITOBA
Scale 1 : 5000000
● Public Water System
▲ Private Water System
Lake Winnipeg
Lake Manitoba
Cedar Lake
Southern Indian Lake
Split Lake
Reindeer Lake
Nueltin Lake
Tadoule Lake
Cormorant Lake
Island Lake
Churchill
Gillam
Lynn Lake
Thompson
The Pas
Grand Rapids
Norway House
Dauphin
Brandon
Portage la Prairie
Winnipeg
Selkirk
Steinbach
Gimli
102°
100°
98°
96°
94°
92°
90°
88°
60°
58°
56°
54°
52°
50°
49°
Cartography by: Surveys and Mapping Branch, Manitoba 1980
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PLANNING URBAN INFRASTRUCTURE REHABILITATION – A RATIONAL APPROACH

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INTRODUCTION

Over the past several years there has been a significant amount of concern expressed about the condition of municipal infrastructure in Canada. In many established urban areas emphasis is now shifting from the design and construction of new systems to the operation and maintenance of existing systems. It is now recognized that there will be a requirement for major capital expenditures to improve the hydraulic and structural performance of sewers and watermains, and this situation is likely to continue into the foreseeable future.

This paper outlines a planning approach to assist engineers and decision makers in determining the most cost effective solution for undertaking these works. Over the last five years this approach has been widely applied and has resulted in substantial savings in the cost of system upgrading.

The method is based on a number of key factors, the most important of which are:

- If major cost savings are to be achieved, and adequate levels of service maintained, a selective program of performance evaluation and upgrading should be initiated.
- The work should concentrate on sewers and watermains where the consequences of failure are the most severe.
- Potential for using cheaper and comparatively new renovation techniques. Until recently there has been some concern particularly with respect to their long term effectiveness but for the more established renovation methods data is now available for all aspects of performance.
- Structural and hydraulic problems are often inter-related and should be considered at the same time. Recent developments in hydraulic analysis of overloaded sewer networks have greatly increased the possibilities of optimising hydraulic performance and assessing the hydraulic implications of possible renovation works.
- Integrated solutions should by developed for all such problems in the study area, including roadways as well as the sewers and watermains. Where possible these should be staged so that the most cost effective solutions can be undertaken first.

Today financial constraints on projects are very familiar to everyone and this approach to infrastructure rehabilitation minimizes the cost of solutions, thereby maximizing the number of problems that can be solved within a fixed budget.

The experience of WRc in the United Kingdom is that the money available for infrastructure renewal can be stretched a lot further by using this comprehensive approach to the problem rather than just jumping in with "the big fix". In other words it is just as important to determine which ones to fix, and when to fix them, as it is to know how to fix them.

THE PLANNING APPROACH

The approach which is outlined in this paper answers the three basic questions:

- which pipes should be investigated?
- which pipes should be rehabilitated?
- which pipes should be rehabilitated first?

As with most engineering problems the best answers are only found after a methodical, rigorous approach, careful investigation and informed decision making.

Our approach has four phases:

- Phase 1: Planning the Investigation
- Phase 2: Assessing the Structural and Hydraulic Performance of the System
- Phase 3: Developing the Rehabilitation Plan
- Phase 4: Implementing the Plan

PHASE 1: PLANNING THE INVESTIGATION

In planning the investigation the initial step must be to determine the objectives of the program. For instance:

- Does the Municipality have major problems in one area and Council has decided that the whole area should be rehabilitated.
- Or, is the Municipality rebuilding roadways and want to ensure that these will not have to be disturbed within the next few years to fix underground services.
- Or, has Council assigned a certain amount of money to be expended throughout the Municipality for infrastructure renewal and requires that the most essential works be undertaken first.

Clearly to meet the objectives of any program, the establishing of a common framework of performance criteria and rules for decision making designed to promote efficient and equitable decision making throughout the area are essential.

The basic questions that need to be answered are:

- What are the critical system components?
- Which pipes need to be inspected?
- Do pipes require upgrading or can problems be solved by reducing hydraulic loadings?
- Which pipes should be upgraded?
- Which pipes require work first?
- If upgrading is necessary should the pipe be renovated or replaced?

Before these questions are answered they require adequate knowledge of the system, field investigations, computer modelling and informed decision making. The key to informed decision making is to use realistic performance criteria for the sewers and watermains and clear simple decision rules.

The performance criteria will be used throughout the infrastructure project and will establish what levels of service are the absolute minimum acceptable, below which a decision will be made to rehabilitate the pipe link, together with a level of service which would be considered desirable for the upgraded system. These performance criteria will include such factors as structural integrity, hydraulic capacity, pressure, etc.

For each of the aspects of performance identified for the different infrastructure components there are three levels to be considered.

- a minimum level which is considered unacceptable and acts as a trigger for the need to upgrade;
- an intermediate target level for upgrading the existing system,
- a 'new scheme' design level that would be achieved in new works.

These performance levels would usually be established by the municipality. Generally, it is not practical or cost effective to establish the target level at a 'new scheme' level. An intermediate level is required and this intermediate target level should be variable for a given performance criteria.

The initial phase of the work is primarily concerned with the tactical decisions about where and when to devote investigational resources within the study areas. It will necessitate planning activities relating to the following sub-tasks:

- System Records
  Central to the planning of the rehabilitation of the urban infrastructure is to know what exists and in what condition it is. If sufficient information is available some form of record upgrading may be necessary - upgrading can be selective, less detailed records would be acceptable for the non-critical areas.

- Critical Links
  The consequences of the failure of a sewer or watermain can vary dramatically depending on the depth, size and location of the link and the proximity of other utilities. In this context 'cost' can be related to the civil engineering cost to repair the sewer and watermain (a direct cost) or the cost of traffic delay, business disruption, etc. (a social cost). Work carried out in the United Kingdom by WRc has revealed that the cost of traffic delay, a cost that can be measured, can at least equal the cost of the civil engineering works. In particularly busy locations the traffic delay can swamp all other costs. It is cost effective to pre-empt failure in these most critical areas, whilst non-critical sections may be ignored and dealt with on a collapse - repair basis. In cases where there are not sufficient monies to rehabilitate the whole system, which is generally the case, it is most important to identify those critical links in the system so that the available monies can be first spent on those critical lengths which promise the greatest cost benefit.

- Performance Problems
  Performance problems should be assessed and identified. Has there been surcharging in the sewer? Has there been flooding? Is there a water quality problem? Is there a history of watermain breaks? Is the water pressure adequate?

- Determine Level of Investigation
  If the more critical sections are, or are suspected to be overloaded hydraulically or undersized for the demand, or if structural upgrading is likely to be constrained by hydraulic problems then a full investigation including system modelling is likely to be justifiable. If however, hydraulic constraints can be confidently excluded then an abbreviated investigation, which would not include system modelling, can be assumed for

planning the investigation.

The key to effective upgrading is to fully understand the existing system performance.

PHASE 2: ASSESSING THE STRUCTURAL AND HYDRAULIC PERFORMANCE OF THE SYSTEM

The second step in the strategic planning would be to make a broad assessment of the problems.

Structural Assessment

In the case of sewers all records of known flooding, pollution or collapse problems should be colleted. If possible these should cover both the perceived causes and the extent of the problems.

The structural condition of the sewer system should then be established. This will normally necessitate planning and undertaking inspections (by CCTV and/or man entry survey) of appropriate sewers. The actual sewers surveyed will hinge on decisions made about performance criteria, sewer criticality, and availability of existing information. In general one would expect to survey only the critical sewers. Following on from the inspections will be the assessment of results to establish risk of collapse or further deterioration. This in turn will enable identification of individual sewer lengths in need of structural upgrading.

The structural assessment of sewers is generally accepted as a function that relies heavily on engineering judgement and experience. It is not generally possible to predict precisely when a sewer will fail. However, knowledge of failure mechanisms allows us to broadly define the risk of collapse, depending on the nature of the defects. As a result a system has been developed which classifies sewers into one of five condition grades which imply different levels of risk of collapse.

Extensive investigation into the deterioration of cast iron watermains has shown that their condition may vary greatly within very small distances. Thus to adequately describe the condition of a 100m length of watermains would be extremely expensive. It has also been shown that there is a poor correlation between the measured condition of the pipe and its performance in terms of failure rate, inadequate pressures, etc. The most reliable indication of the future performance of a pipe is its past performance. Thus the recommended approach centers on measurements and records of performance (break history, pressure surveys and hydraulic models, etc.) rather than measurements of the condition of the pipes.

At a certain break rate, it will become more economical to replace the main than continue to repair it. An economic analysis will determine the level at which this occurs.

Another performance criteria for watermains is the aesthetic water quality. Discolouration and taste of water may have a number of causes, including internal corrosion of pipes. The definition of aesthetic water quality standards and corresponding method of measurement can be difficult and often quite subjective, especially when involving taste.

Hydraulic Assessment

One of the most important factors in the overall planning approach for sewer rehabilitation is the building and verification of a computer based model of the collection system under consideration.

For an adequate analysis to be achieved the model must cover the whole of the catchment. It is not necessary to model the entire catchment to the same level of detail however. Investigations are concentrated on the critical sewers and interconnecting pipelines and it is this area, termed the core area, which must be modelled in detail. Modelling of the area outside of the core is only necessary to provide realistic inflows to the core area and this section of the model may therefore be simplified. This enables computer capacity to be used most effectively and reduces the cost of the analysis.

Once the model is built the accuracy of its predictions must be verified by a well designed flow survey. Depth and velocity measurements are taken at strategic locations and collated with local rainfall measurements. Comparison of measured flows with computer predictions will usually highlight deficiencies in the model and will suggest areas of survey which should be checked. Errors in the input data may then be corrected without the need to resort to "force fitting" of the model.

Once verified in this manner the model can be used with the confidence that it realistically represents the system and will predict flows to an acceptable level of accuracy. The importance of attention to detail and technique in the modelling and verification process cannot be overemphasized.

The verified model is used to quickly identify problems in the system such as:

- flooding due to a lack of capacity can be differentiated from flooding caused by backing up from a problem further downstream,

- inadequate capacity due to siltation or pipe collapse can be differentiated from lack of capacity due to undersizing of pipes.

The hydraulic performance of a water distribution system is also assessed using a computer model. Most municipalities establish minimum design standards for their distribution system under certain demand conditions, e.g. pressures below 40 psi during peak hour demand are unacceptable, or below 20 psi for maximum day demand plus fire flow are also unacceptable.

In the terms of the cost effective planning of rehabilitation work and the future management of the system it is highly desirable to have an accurate water network analysis. This is particularly important when the distribution system needs to be redesigned to accommodate future development or where the existing system is inadequate to meet even the present demands. Such information can aid the decision making process when it comes to the evaluation of the alternatives of relining or replacement.

PHASE 3: DEVELOPING THE COST EFFECTIVE REHABILITATION PLAN

Objectives and Approach

The major objective is to develop one or more sets of rehabilitation proposals each of which meet all the identified needs of the system and then to select the most cost-effective solution. Each proposal should satisfy the following criteria:

- economy in construction costs,
- minimization of disruption during construction,
- ease of staging the works according to the priority attached to different problems.

Structural Performance Problems

Having established a list of priorities for each sewer or watermain with unacceptable structural condition, establish:

- whether renovation could offer an appropriate solution,
- the approximate cost of that renovation,
- the approximate capacity of the sewer or watermain after renovation,
- the cost of renewal where renovation is impracticable or as an alternative where renovation only appears to offer small savings in construction costs or disruption.

Hydraulic Performance Problems

Drainage System

The hydraulic model is used to formulate solutions to the hydraulic performance problems identified in Phase 2 and to assess what the hydraulic performance of the system would be following completion of all the probable renovation and renewal works. Lengths that are then in breach of the hydraulic performance criteria (upgrading target) are thus identified. Where alternative measures offer significantly different capacities, the system performance should be assessed for each option.

Develop Outline Plans

Drainage System

The probable measures for dealing with the structural rehabilitation have been identified. When planning the upgrading works to relieve hydraulic problems covering flooding, surcharge and pollution the first steps are to incorporate the probable measures for structural rehabilitation and to make allowances for any future development which may be planned.

The solutions to hydraulic performance problems to combined sewers could be considered under four specific headings:

- reduce hydraulic inputs to the sewer system, e.g. through sewer separation,
- maximize use of existing sewer system capacity, e.g. by removing throttles or relocating throttles,
- attenuate peak flows by on-line or off-line storage,
- increase sewer system capacity by replacement with larger capacity sewer or reinforcement with additional sewers.

Water Network

In a similar manner to the drainage system, when planning the upgrading works to relieve hydraulic problems the first step in sound cost effectiveness planning is to incorporate the probable measures for structural rehabilitation.

Watermains in good condition which are identified for upgrading for hydraulic reasons may be left intact and an adjacent main in poor condition may be oversized to supply the needed flows and pressures to the adjacent link.

In developing proposals for rehabilitation, the ideal should be to achieve a level of performance equal to the Upgrading Target Criteria. However, in some systems, considerable improvements in performance may be attainable at a cost significantly less than that necessary to attain the full target for upgrading. The Municipality must then re-assess the targets in the light of the economics of the situation.

Priorization

In order to set priorities for rehabilitation, account should be taken of each of the performance aspects that have been evaluated. The priority attached to a particular problem has two aspects, its severity and its extent. The severity of a problem is most readily measured in terms of levels of performance previously established. The extent of the problem can relate to its length, area covered or number and frequency of properties affected, etc.

A severe but localised problem will merit a different level of priority to a less severe but extended problem. To enable priorities to be set for each utility and then to integrate these priority levels across the utilities a rating method must be introduced.

Integration

Priorization on a block by block [infrastructure links] basis for the rehabilitation project can be evaluated by the summation of each components overall rating. It is possible to introduce a weighting factor for each component to reflect the importance that the Municipality attaches to each component.

The infrastructure links which have the highest numerical rating, together with those links, identified as having a "trigger" for rehabilitation will comprise the rehabilitation project.

With the preferred overall solution to problems identified, the final planning step involves the timing of the various works proposed and the requirements for future monitoring of the structural condition of critical sewer and watermains.

PHASE 4: IMPLEMENTING THE PLAN

This phase is essentially the implementation of the rehabilitation area plan and the monitoring of the system to check that the plan remains appropriate.

The design and construction of the works recommended in the earlier phases will be undertaken in accordance with the timing established in the approved plan or as subsequently updated in the future or as allowed by the availability of capital funds.

The structural condition of the critical links should be regularly monitored to determine whether the previously assessed rehabilitation needs and priorities remain correct. Changes to the critical condition ratings must be noted and amendments to the plan made as the changes in conditions warrant.

The system records and hydraulic models must be kept up to date so that they continue to represent the system in its current form.

Where events occur or circumstances change so that the plan is no longer a satisfactory basis for future work, the plan should be reviewed and updated as necessary.

RENOVATION TECHNIQUES

Renovation involves upgrading by methods which utilize the existing fabric of the sewer or watermain. Since the existing pipeline is not removed and replaced, renovation can be carried out without the extensive open-cut excavations normally associated with pipeline rehabilitation works.

Renovation techniques are still relatively new but several techniques have been in use long enough for problems to be identified and resolved. WRc has carried out extensive research into these techniques in the United Kingdom and has developed design methods, specifications and quality assurance procedures which enable them to be applied with the confidence that pipeline performance can be upgraded to acceptable standards for a service life of at least 50 years.

Different renovation techniques have been developed for various combinations of performance deficiencies and installation environments.

In the context of renovation works, sewers may be considered in three broad categories:

1. Man-entry, greater than 900 mm high, to which men can gain entry and work.
2. Man-accessible, between 500 and 900 mm high, to which men can gain entry, but would be unable to work effectively.
3. Non man-entry, less than 500 mm high, which men would not be allowed to enter.

Water mains would normally be considered as non man-entry.

Some renovation techniques may be applied in all categories, whilst others are limited to just one or two.

The established techniques fall into three distinct groups:

1. Stabilization, which involves the repointing of joints in brick sewers or the chemical injection of the surrounding ground through defective joints in the pipeline.
2. Lining, which involves the installation of a liner either in segmental form or as a continuous pipe.
3. Coating, which involves the in situ application of a new internal coating.

Stabilization techniques are normally only applicable to sewer renovation. Stabilization may be adopted where the sewer is still physically intact but has suffered deterioration at joints. Repointing is only relevant to man-entry brick sewers, but chemical grouting may be applied to all categories of sewer. The only available chemical grouting system for non man-entry pipelines is patent protected, however.

Lining techniques may be carried out using a variety of materials including glass reinforced plastic, polyolefine (polyethylene or polypropylene), fibre reinforced plastic (Insituform), glass reinforced cement, resin concrete and precast gunite.

Linings may be installed in segmental form (limited to man-entry and man-accessible pipelines), as complete pipes (sliplining), or, in the case of Insituform, as a flexible resin-impregnated tube which is cured after insertion.

The only established coating technique is In Situ Gunite which, by its nature, is limited to man-entry pipelines. Development of other spray coating techniques which may be applied in non man-entry situations is currently underway, but these systems are as yet unproven.

In view of their potential for widespread use in Canada, the following three techniques are worthy of further explanation:

1. Insituform: This involves the insertion into the sewer of a flexible, resin impregnated tube which, once in position, is cured to form a close-fitting lining. The tube is inserted by inversion under a head of water and is heat cured by raising the temperature of the water.

   Insituform is installed through existing manholes and is therefore relatively quick to apply; it is also hydraulically efficient since the close fitting liner minimizes the reduction in cross-section of the sewer. Flows must be diverted during installation and curing of the liner however.

   Service connections may be made from within the sewer for most sizes of pipeline by utilizing a remote cutter.

   Due to the nature of the materials used, Insituform is not suitable for watermain renovation.

2. Sliplining: Sewers and watermains may be sliplined using polyethylene or polypropylene pipes. The pipes are drawn and/or jacked into position in continuous strings and the annulus between old and new pipelines is grouted. Adequate grouting is crucial to a successful sliplining operation and it is therefore essential that grouts are carefully specified and grouting is carried out under controlled conditions.

   Traditional sliplining uses polyethylene pipes in 6m or 12m lengths jointed by butt fusion welding. Pipe jointing may be carried out at ground level, in which case a lead-in trench is required to enable the liner to be "snaked" into position, or at invert level, in which case a short section of open trench is necessary to enable additional pipe lengths to be added to the string.

   Recent developments have produced pipes with mechanical joints. For ease of jointing these pipes are supplied in short lengths (typically 500mm) and in such lengths they may be installed via existing manholes. This involves less surface disruption but, due to the greater number of joints to be made, insertion is considerably slower than with fusion welded joints.

   Normally all flows would be overpumped whilst sliplining is being carried out but, in exceptional circumstances, nominal flows can be allowed to flow through the annulus between the existing sewer and the lining. Flows must be controlled however to ensure the safety of operation and to prevent rags and solids becoming lodged in the annulus to the detriment of the grouting process.

   At present there are no systems for the remote reconnection of laterals and service connections. This must therefore be carried out by external excavation.

3. Percussive Moling: Percussive moling was developed jointly by the gas and water industries in the United Kingdom. It has been applied extensively to gas main replacement and is now becoming established in water and sewer applications.

   The technique involves passing a percussive mole through the pipeline breaking up the existing pipes and consolidating the surrounding ground. The void thus formed is sliplined as the mole proceeds. Ground relaxation then normally occurs and soil loads are transferred to the new pipeline.

Strictly speaking, moling is a replacement technique rather than renovation, since the original pipe is destroyed and replaced by a new one. It retains the existing "hole in the ground" however, and therefore has more in common with modern renovation techniques than with traditional replacement methods.

Moling provides a non-disruptive method of replacing pipelines which are deformed beyond the 10% limit of the other renovation techniques. It also allows pipelines to be replaced with larger pipes if hydraulic considerations so require.

The ease or difficulty of a particular moling operation is determined by the combination of many factors. Principal among these are ground conditions and the nature of the existing pipeline.

In general terms, moling becomes less difficult as ground conditions change from sands and gravels through to stiff clays. A knowledge of the ground conditions is therefore important when considering whether to adopt the technique and essential to Contractors tendering for moling work.

With the exception of ductile iron, most common pipeline materials can be replaced by moling. Progress may be slowed considerably if ductile iron fittings or concrete surrounds are encountered on the pipeline; it is generally advisable to locate and remove these prior to the moling operation.

One further aspect of moling which should be considered is the effect on adjacent structures and services. As would be expected, ground movement is greater for pipe enlargement works than for size-for-size replacement and the effects are transmitted further in firm ground than in soft. Current WRc studies indicate that size-for-size moling should not be undertaken within 0.75 m of sensitive services or structures. This minimum separation should be increased to 1.0 m if pipeline enlargement is being carried out.

CONCLUSIONS

This paper has described a logical approach to the task of dealing with the complex problem of devising and executing a cost effective plan for the upgrading of urban infrastructure. Considerable experience is already available in the approach as applied to sewer and storm water systems and to potable water systems. However, the combination of this with the needs of the road system upgrading in an urban area is relatively recent. The first application of this is in the City of Edmonton where substantial benefits in terms of cost savings and reduced disruption to the community will be derived.

ALUM CLARIFICATION & CONTROL

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INTRODUCTION

It is perhaps interesting to note that aluminum salts have been used for water clarification at least since Roman times, and it is a fascinating thought, given early Roman management techniques, to consider what might have happened to their operating people when their floc blankets got away on them.

In this paper, we thought we should briefly review common types of water treatment equipment, discuss their idiosyncrasies and perhaps develop some sensible guidelines to assist the operator.

EQUIPMENT

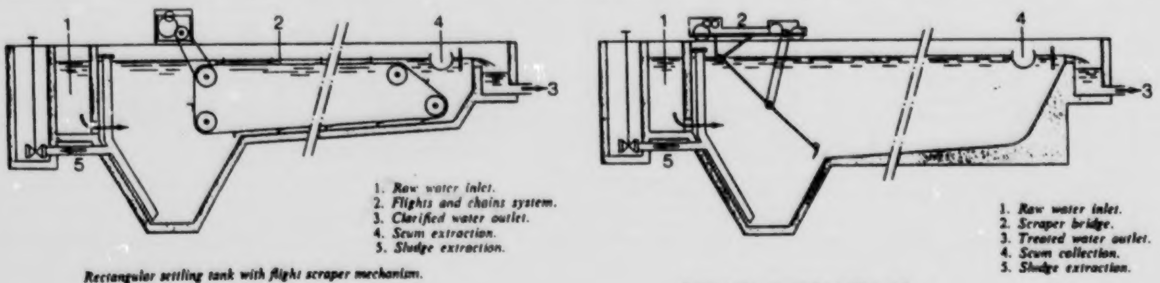
Like any other machine, there has been an evolution, of sorts, in the development of alum clarifiers. The first, of course, was the well known "hole-in-the-ground" or settling tank. These are static settling systems, which started out looking like (Figure 1) large, crossflow type units which incorporated separate rapid mix and flocculation sections. While many of these are in use today, they did tend to suffer from the disadvantages that -

a) They required quite large areas.
b) They did a very poor job of sludge thickening.
c) Their sludge collection systems ranged from batch - i.e. once per year cleanout, to continuous, which involved underwater rake bearings with sludge hoppers at one end, or travelling bridge types which were quite expensive and often a maintenance headache.
d) Their sludge recirculation was severely restricted and poor solids contact could be achieved.

FIGURE 1
Rectangular Settling Tanks

Rectangular settling tanks have the advantage of a more compact layout of the various treatment units, but on the other hand are generally more costly. A $\frac{\text{length}}{\text{width}}$ ratio between 3 and 6 is generally used. Tank depth is usually between 2.5 (8′ 6″) and 4 m (13′ 1″). The slope of the bottom is about 1 %.

The scraper system may be driven either by a bridge spanning the tank and moving from one end to the other or by endless chains under the water.

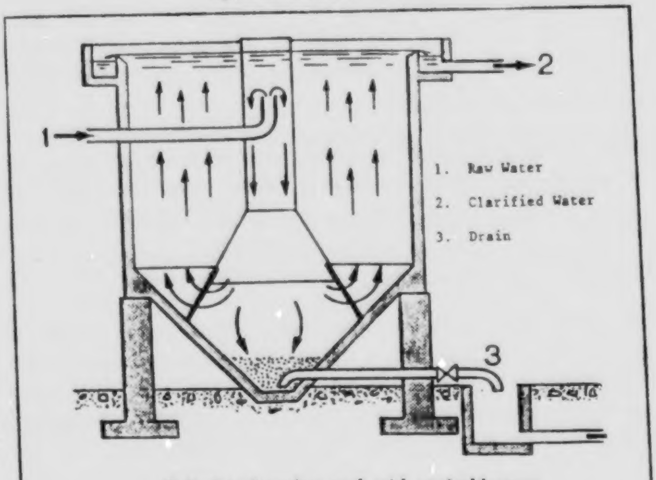


Rectangular settling tank with flight scraper mechanism.

Scraper-type rectangular settling tank.

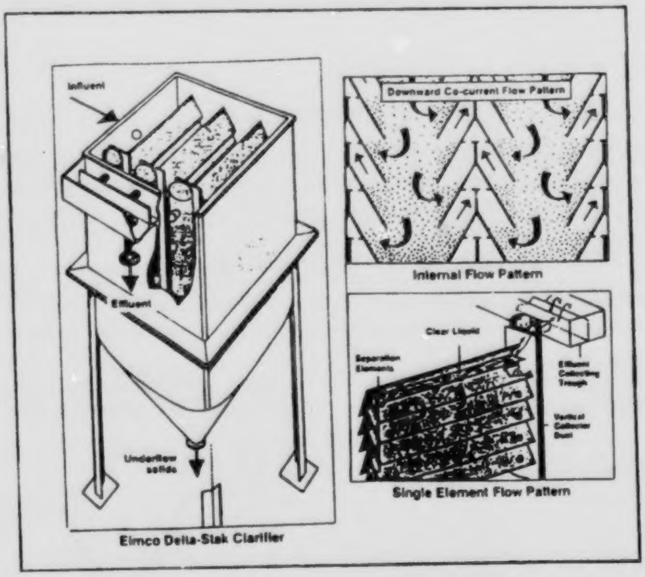
The **Upflow Settling Tank** (Figure 2) is a pretty basic piece of equipment, and it suffers from the same problems of sludge handling and thickening as the crossflow type. With the addition of plate separators, tube settlers, etc., these can, at least, be reduced in size to handleable dimensions, and some now have live (raked) bottoms which assist the sludge collection system (Figure 3).

FIGURE 2
Upflow Settling Tank


1
2
3
1. Raw Water
2. Clarified Water
3. Drain

Cylindrical settling tank with conical bottom.

FIGURE 3
Upflow Settling Tank with Raked Bottom


Influent
Effluent
Underflow solids
Downward Co-current Flow Pattern
Clear Liquid
Separation Elements
Effluent Collecting Trough
Vertical Collector Duct

Internal Flow Pattern

Single Element Flow Pattern

Elmco Delta-Stak Clarifier

Then there is the **Solids contact, clarifier**, which comes [Figure 4] in many forms. Shown here are:

1. The **Peripheral Feed Clarifier**, made by Envirex Corporation.

2. The **Dorr Oliver Hydrotreater**, which featured a bottom distributed feed system.

3. The **Walker Clariflow**, which contained the early elements of centre feed, mechanical recirculation clarifiers, although the "propeller was offset and high tip speed".

4. The **Dorr Oliver Pretreater**, which used external rapid mix and external sludge recirculation using centrifugal pumps. This is not usually recommended when fragile floc is being handled, for obvious reasons.

5. Recognition of the problems connected with pump shear on Aluminum Hydroxide floc, led to the development of [Figure 5] clarifiers such as the **Pulsator**.

FIGURE 4
Solids Contact, Clarifiers

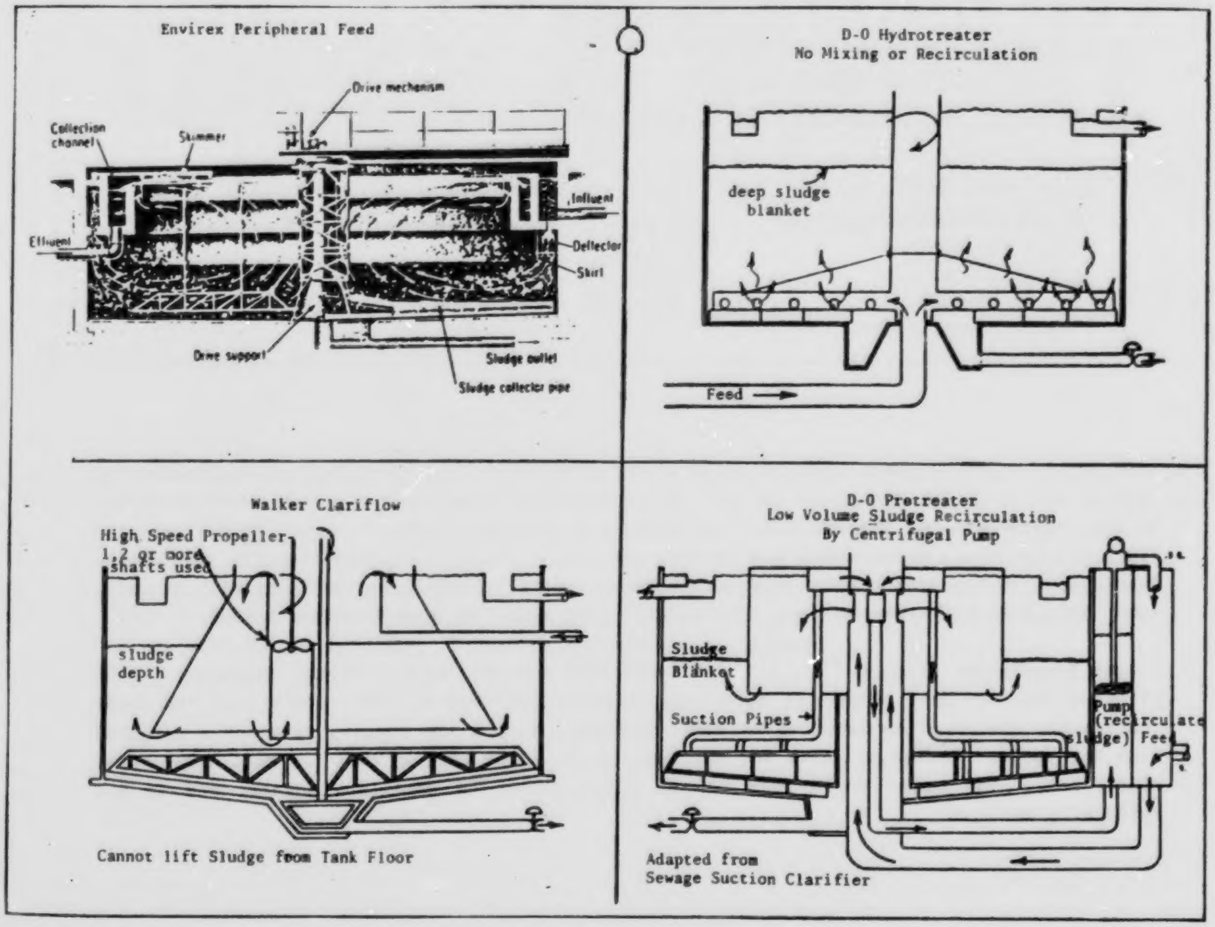
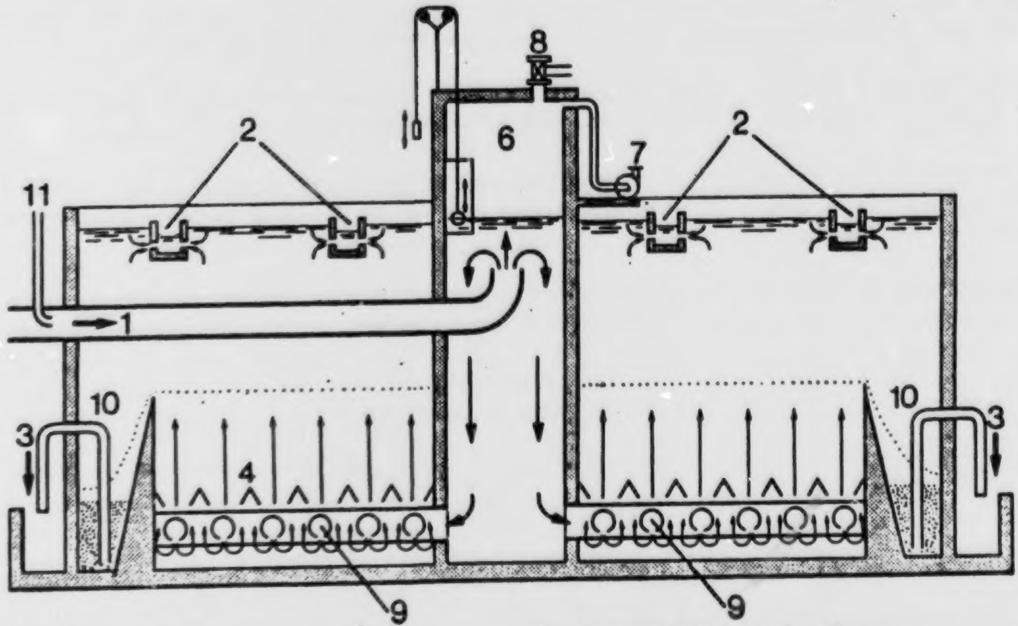

Envirex Peripheral Feed
Drive mechanism
Collection channel
Skimmer
Influent
Effluent
Deflector
Skirt
Drive support
Sludge outlet
Sludge collector pipe
D-O Hydrotreater
No Mixing or Recirculation
deep sludge blanket
Feed
Walker Clariflow
High Speed Propeller-
1,2 or more shafts used
sludge depth
Cannot lift Sludge from Tank Floor
D-O Pretreater
Low Volume Sludge Recirculation
By Centrifugal Pump
Sludge Blanket
Suction Pipes
Pump (recirculate sludge)
Feed
Adapted from
Sewage Suction Clarifier

FIGURE 5
Pulsator

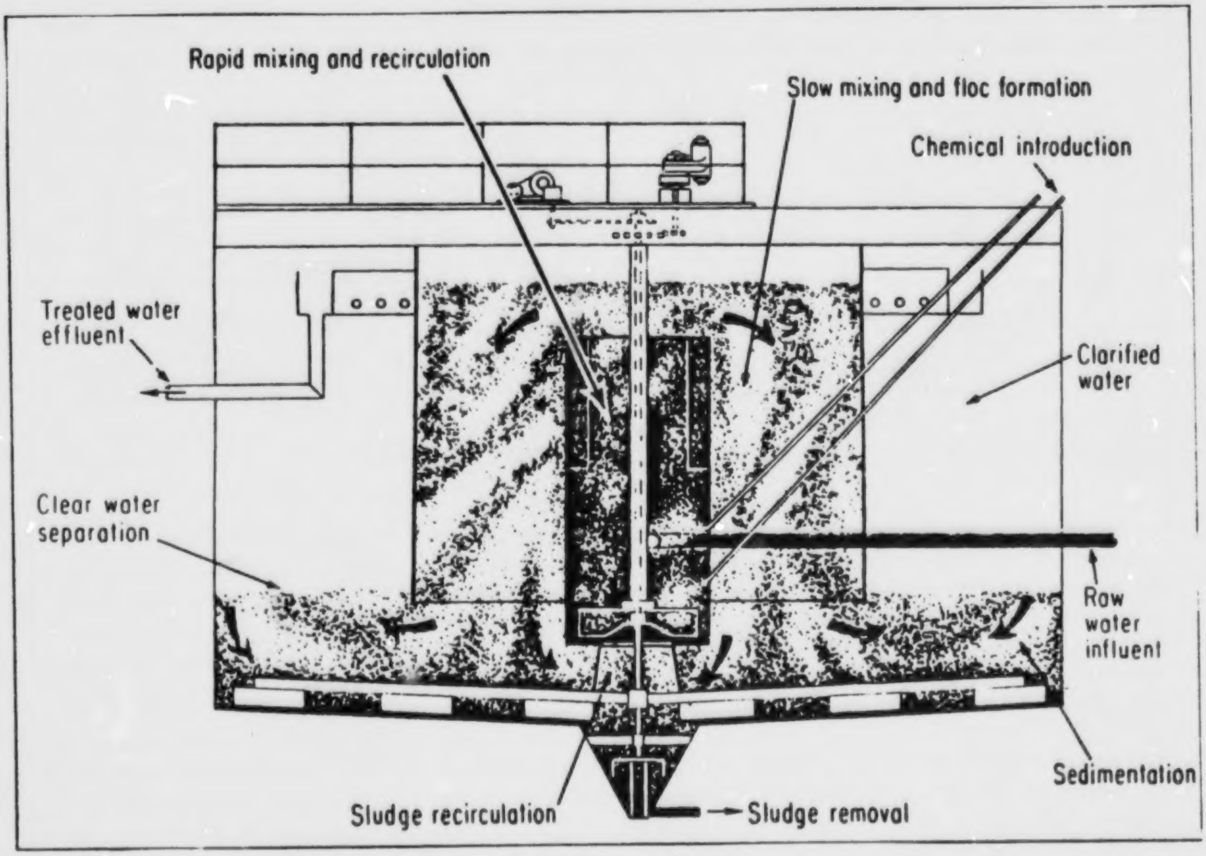


1. *Raw water inlet.*
2. *Clarified water outlet.*
3. *Sludge discharge.*
4. *Stilling plates.*
6. *Vacuum chamber.*
7. *Vacuum pump.*
8. *Automatic vacuum-breaker.*
9. *Raw water perforated distribution piping.*
10. *Sludge concentrators.*
11. *Reagent inlet.*

This is a solids contact unit which does not provide any mixing energy, but operates by raising and lowering the level of the inlet chamber, using a vacuum pump, and providing bottom distribution of the raw water and chemicals into the sludge blanket. This treats the floc very gently and provides considerable solids contact. The deficiencies tend to be connected with clarifying water with quantities of silt, or where the sediment will compact, since they have no bottom desludge connection. They also have poor turndown ratios.

6. **Graver Reactivator** (Figure 6). This unit can take several configurations depending on the point of application. Generally speaking, it has a cylindrical lower draft tube and outer draft tube, and impeller tends to be small diameter, with a low pumping rate and a somewhat high tip speed. Design criteria for the tank is about the same as that for most other water clarifiers. The outer draft tube can be quite large, is sometimes square, and is not internally baffled.

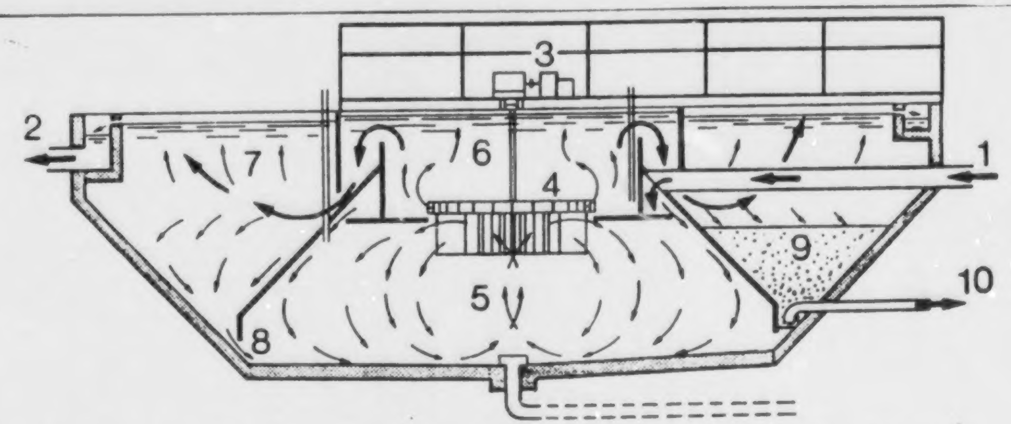
FIGURE 6
Slurry Recirculation Design Clarifier


Rapid mixing and recirculation
Slow mixing and floc formation
Chemical introduction
Treated water effluent
Clarified water
Clear water separation
Raw water influent
Sedimentation
Sludge recirculation
Sludge removal

7. **Infilco Accelator** (Figure 7). This unit was originally rakeless, employing a static bottom with several desludge points and a steep wall portion on the bottom to guide the sludge to the desludge points. The unit has a primary and a secondary mixing zone, employs a relatively large diameter paddle, which is set high in the clarifier and has a relatively low tip speed. It is somewhat more of a solids contact unit because of the design of the primary and secondary mixing chambers. The primary hood is conical and sludge/water mixture is circulated to the secondary (flocculation) zone by the paddle.

FIGURE 7
Infilco Accelator

The Accelator NS clarifier has a central reaction zone surrounded by a settling zone. These two zones communicate with each other at the top and bottom.



1. *Raw water inlet.*
2. *Clarified water outlet.*
3. *Impeller drive.*
4. *Rotor impeller.*
5. *Primary mixing and reaction zone.*
6. *Secondary mixing and reaction zone.*
7. *Clarified water.*
8. *Sludge return.*
9. *Sludge concentrator.*
10. *Excess sludge discharge.*

8. **Eimco Reactor Clarifier** (Figure 8). For obvious reasons, we are more familiar with this unit, and can point out that the design features include:

   a) The large diameter, low tip speed, high volume recirculating turbine.

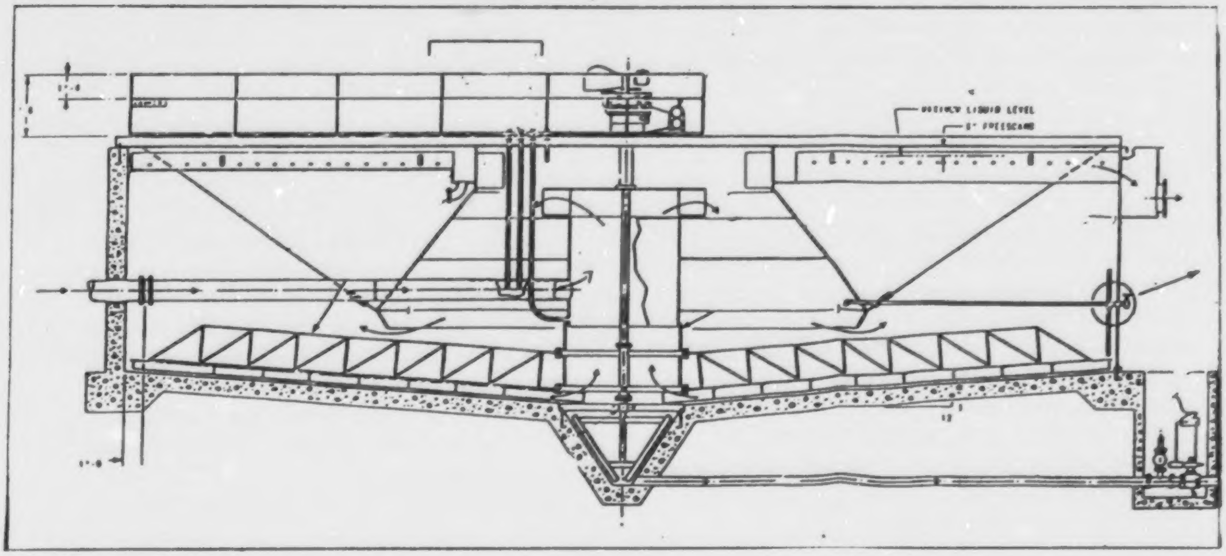
   b) Recirculation drum, which includes a rotating portion to insure that solids are sucked from the bottom of the clarifier. Note: that the turbine is set high in the clarifier to ensure that the turbine is not spinning around in the sludge blanket itself.

   c) A large diameter conical hood, internally baffled to act as a stilling well or energy absorbing feedwell; since it is necessary to remove the energy input of the rapid mix zone mix, i.e. the turbine, to achieve solids/liquids separation. Most precipitation takes place under the hood.

   d) A rake drive system which is high torque, controlled by an electro-mechanical torque meter. This generally tends to be somewhat overdesigned, but on the other hand, the main drive tends to last a lot longer when it is not being worked towards the high end of its torque capability. Sludge is removed from the centre bottom after being thickened by both the rake and the bottom thickening pickets.

FIGURE 8

Eimco Reactor Clarifier



In general terms, all upflow clarifiers share many of the same characteristics. All must have a rapid mix section, which is normally combined with the internal sludge recirculation, solids contact function using the internal recirculating turbine. Standards vary for this, but while we tend to design for a velocity gradient of better than 700 sec -1, many clarifiers in the field operate quite successfully in alum clarification with velocity gradients as low as 250 sec -1. (Velocity gradient, as you know, is defined as the energy input into a liquid of specific viscosity for a given period of time).

The rake torques again vary, so that our standard for rake torque for alum clarification is in the region of $2D^2$ to $3D^2$ with some clarifiers operating again quite successfully with numbers considerably less than this. Since torque measurement is one of the control parameters for any clarifier, we feel that it is important that the total torque capability of the machine be known. Generally, unless the raw water contains quantities of silt, alum clarifiers require little torque (Table 1).

TABLE 1

Alum Clarifier Standards

	Residence Time	Velocity Gradient
Recirculating Drum	60S	700S-1
Reaction Well	15.3 m	20 - 40S-1
Rise Rate	0.75 USGPM/Ft.^{2}	(1.25 USGPM/Ft^2 with tube settlers)
Rake Torque	2 - 3 D^2	

The flocculation zone of the reaction well is again common to all alum clarifiers. Standards vary from residence times of 15 to 30 minutes, with or without internal baffling, and conical or circular (or for that matter, square) configuration. Our thought is that if the reaction well is to be designed as a flocculating chamber, it should have more or less the same criteria for velocity gradient as the classic flocculating chambers in rectangular clarifiers, i.e. 20 - 40 sec -1 velocity gradient. Again, this is a calculated value which includes the volume of the reaction well, instead of the recirculating drum. We feel that the internal baffling is necessary to provide an energy absorbing environment, so that proper flocculation and hydroxide gel growth can take place.

Launders

These vary depending on clarifier total diameter, size, tube settlers, etc. We feel that the best form of launder system is a radial launder, radiating from the central collection system to the outside edge of the clarifier. This has been extremely successful, and follows generally accepted water treatment plant practice. This is not to say that peripheral launders cannot be used, and both inner annular and outer annular launders have been successfully used, although mostly in the smaller diameter (less than 30') clarifiers. We have successfully used an inner annular radial launder design, on clarifiers up to 65' diameter and many clarifiers operate with launders which are square surface pipe launders, sub-surface pipe launders, and, for that matter, outer annular pipe launders in tankage as large as 65' diameter. Again, it is generally accepted that if tube settlers are in use, a radial launder system is necessary in any clarifier of greater than 30' diameter.

ALUM CONSUMPTION AND DOSAGE

Clearly, the only way to establish alum requirement is by jar study. This is a standard AWWA or TAPPI test which involves adding alum in varying dosages, usually in increments of 5 or 10 ppm to a series of six samples of the same water, allowing a two minute rapid mix and a ten minute slow mix, to determine the minimum level of alum which would permit coagulation and clarification to occur. This is a relatively simple test, and anyone operating in a river system should have the chemical solutions available at all times to be able to perform a jar study should the river take a sudden change in turbidity or colour (Figure 9). While there are no absolutes, it can be stated generally that alum dosage requirement increases as turbidity increases, is relatively high for colour removal and is relatively high for algae removal. Unfortunately, the methods of measuring turbidity, colour and algae have not been related to total numbers of particles, except in some very sophisticated cases. We can probably generalize and state that to achieve complete coagulation, alum dosage is related to turbidity or colour levels and since the requirement is that there should be at least the minimum alum introduced, and a small amount over the minimum really does not do a great deal of harm. Many people, when operating in surface water systems which are subject to rapid change will deliberately overfeed alum during their runoff periods to provide themselves with at least enough coagulant. It is difficult to absolutely correlate turbidity measurements in terms of NTU (or FTU) with alum requirement (Figure 10). It is often site specific, and the prudent operator would attempt to keep very careful records of alum requirements at various turbidity levels in any given river or surface water system, to the point perhaps of graphing the relative turbidity versus alum requirement numbers to provide a ready reference for the next time the system changes. The same can be done, more or less, for colour.

FIGURE 9
Relative Turbidity versus Alum Requirements

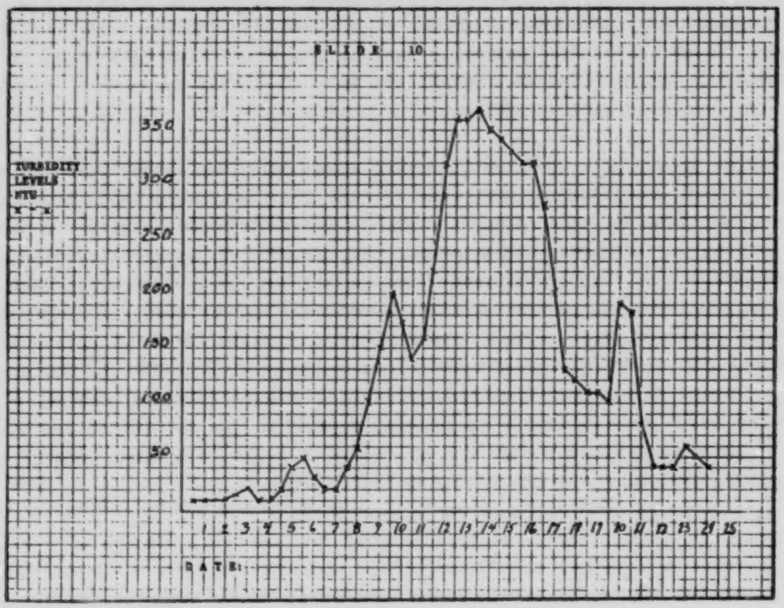
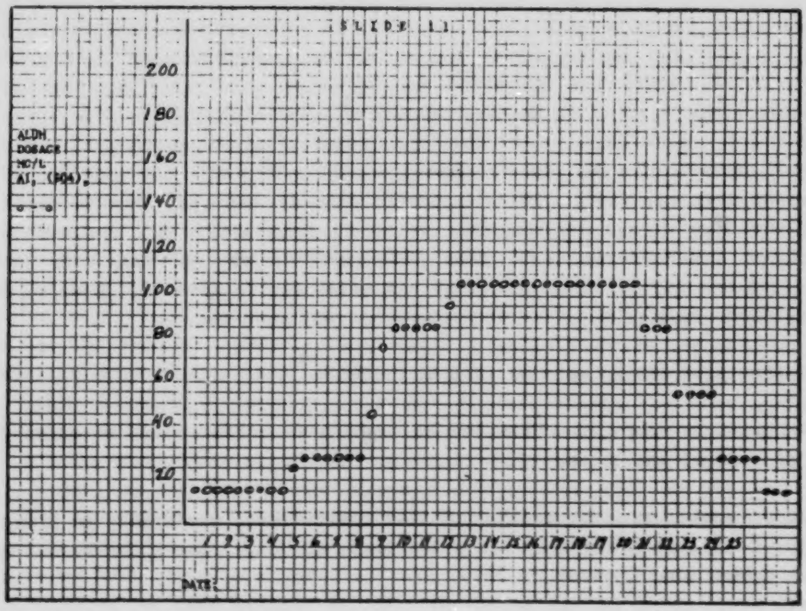


FIGURE 10



Clearly, the higher the alum dosage, the greater the need for pH correction at the backend, alum being an acidic salt. Depending upon location, many people use lime or sodium hydroxide to bring the final pH of the treated water to something above the neutral point of 7 pH, mainly to protect distribution equipment and lines. This, of course, is not the same as for Municipal lime softeners where the pH correction generally must be downwards with recarbonation equipment, etc. Generally speaking, again, lime is cheaper but more difficult to handle, caustic soda is expensive, but can be purchased as a liquid.

In the municipal field, there is a small but growing movement to use other forms of alum salts, such as sodium aluminate or polyaluminum chloride. These materials have their benefits and, for some specific cases, may even be economically viable. The advantages usually are that they are relatively fast reacting in low temperature water, they provide an upper pH correction and some aluminum ion to assist in the coagulation and flocculation process. Depending upon the water, they can sometimes be used alone or accompanied by a reduced feed of aluminum sulphate. Our own experience is that in areas of high colour, low turbidity such as Canadian Shield water, which is often low in total dissolved solids and therefore relatively unbuffered, the usage of some quantity of alum to break the colour colloid, potentially superimposing one or other of the aluminum salts can be a valuable method of producing a neutral pH, well clarified water. Also, clearly the more chemical additives which are in use at a plant, the more complex the control procedures become, and chemical costs can escalate fairly rapidly.

Polymer flocculants with long chain, high molecular weight molecules are widely used in industry, largely because industrial clarifiers often operate close to their maximum hydraulic limits, and control of the sludge blanket can be difficult without chemical assistance. Their use in municipal clarifiers appears to be less widespread, but they can certainly form a very effective method of controlling sludge blanket characteristics.

CLARIFIER CONTROLS

In the final analysis, the water treatment clarifier is an extremely simple piece of apparatus, containing usually only two moving parts, and should not represent any great difficulty as to operation and control, since the operation is dependent only upon alum dosage and sludge inventory. We are often tempted to wonder, therefore, why it is that we find ourselves staring into clarifiers which are producing incredibly poor quality water causing untold problems at the filters and great amounts of grief for the operator, who after all, has something else to do in his day than watch chemical feed. It is a truism that if the clarifier is not hydraulically overloaded, if the alum dosage is adequate, if pH is held within the correct range, and if desludge techniques are properly used, then the clarifier will produce water which will be properly coagulated and flocculated, clarified and eminently filterable. Clearly also, if this is not the case, then one of these four parameters has not been met.

Control systems should be set up more or less as follows:

a] Identify what must be removed. This may be turbidity, colour, iron manganese, algae or a combination of some of the above.

b] Find a method of measuring concentration of that contaminant, preferably in line, but if not, then it should be measured manually every two hours for a surface water system and daily (or weekly] for a ground water system.

c] Set the minimum background alum dosage to provide coagulation for the background level of that contaminant. This is normally done by jar study or experience.

d) Set the detecting instrument to alarm if the contaminant level in the incoming water reaches above a predetermined point.

e) At the alarm point, recheck the alum requirements, either by jar study or experience and reset the alum dosage.

f) Reverse the procedure if the concentration of contaminants falls.

g) Pay a great deal of attention to sludge inventory control. This last bears a certain amount of investigation since sludge is one of the critical factors of operating an alum clarifier and it is also the least understood of all factors.

Sludge is, in the main, a product of what is fed as chemical to the clarifier. The bulk of the sludge in the clarifier is normally aluminum hydroxide with perhaps minor quantities, sometimes of sand, silt, turbidity solids, colour, organic or iron absorbed on the floc particle. Nonetheless, the bulk of the floc comes from the alum being fed. It stands to reason that the more alum that is fed, the greater the desludge requirement will be.

The sludge level in any alum clarifier is somewhat site specific and may change winter to summer, and certainly from flow to flow. The worst condition would be a high hydraulic loading mid-winter with extremely low temperature water, at which point, no clarifier is particularly efficient at separating hydroxide floc from the water and the filters must then be called upon to provide the final screening process. This is not bad, indeed this is why the filters are there. Most municipal filters are designed to take anywhere from 10 - 20 ppm of overflow suspended solids and properly remove it to provide a sparkling clear effluent. Most operators prefer to see the filters receiving less than 1 NTU turbidity water, because this means that they are not loading the filters in any way, run lengths as high (or backwashes infrequent) and filter media remains clean. Nonetheless, it is worth repeating that it is well within the design parameters of municipal filters to receive suspended solids of up to 20 ppm. (This is not JTU turbidity but suspended solids). If the particles passing through the filters are in the form of turbidity, then the problem lies in the clarifier and the alum dosage, and the filters will not filter out turbidity solids anyway.

(Figure 8) So, where to keep the level of sludge in the clarifier? The answer, of course, is that it depends. If the clarifier is at the upper end of its hydraulic loading, at about 1.25 gallons per minute per square foot, and if it is not fitted with tube settlers and if it is subject to fluctuating load conditions, then it can probably reasonably be said that the clarifier must operate with effectively zero sludge blanket or zero sludge inventory. Conversely, if the clarifier has a relatively low hydraulic loading (less than 0.8 GPM/Ft.^{2}) if it is fitted with tube settlers, if it is base loaded, then considerable quantities of sludge can be maintained inside the clarifier, and while I would not recommend this, I have seen clarifiers operating with sludge levels effectively up to the launders while still producing extremely good quality water. The correct sludge blanket levels for most upflow clarifiers under the circumstances would be at a level which is approximately just above the bottom of the flared portion of the hood, (or just above the outer draft tube lower lip in the case of some Ecodyne units). But! as anyone who has operated a clarifier with an extremely high sludge blanket knows, any upward change in the clarifier loading will result in the total expansion of the sludge blanket and cause considerable quantities of alum solids to flow into the filters. This is generally because the alum blanket at this point is already into the hindered settling range, and any increase in flow expands it by a factor of two to three times its volume. If this happens while the sludge blanket is being held at lower levels, then at least there is some space in the clarifier for the blanket to expand. If, however, you are in the situation where you have shut the clarifier down for any period of time, and the sludge blanket has effectively dropped to the bottom of the clarifier, don't be surprised if it blows over onto the filters for a while when you get started up. These blankets are held in a dynamic equilibrium against the upflow of the water, and the disruption of startup causes all sorts of problems. I would suggest very slow startups, i.e. 0.1, then 0.2, then 0.3 GPM/Ft.^{2} over 40 minute periods.

Desludge rates should be sufficient to maintain the sludge blanket at the "comfort" level but please note that desludge rates can vary from 60 seconds per hour to 60 seconds per day, depending on the total amount of alum being added and, on the characteristics of the clarifier, thickening capabilities, etc. If you prefer to run a clarifier as a sludge blanket unit, you may wish to invest in some sampling device which permits you to determine the height of the sludge blanket in the clarifier. There are some commercial units available, or some specialty variable depth sampling devices which can permit detection of sludge blanket level.

Careful notes taken over several days, can provide a measure of increase or decrease in sludge blanket level, so that appropriate action can be taken at the desludge valve. While it is not as important if the clarifier is not being used as a sludge blanket unit, the desludge line should be able to be sampled, since only by having a look at this material is it ever possible to determine the quality of the sludge in the clarifier.

SAMPLING

There are several sample points for any given clarifier. I don't suppose it would be much surprise to anyone to hear that our sample points are seldom used as control points, and that if they were, the unit would probably be overtorquing anyway. Sampling alum floc is not a simple procedure since the pipe sampling point which we provide, and which are really designed for lime softening controls, are quite inefficient in that there is enough inlet velocity to shear the floc and enough turbulence along the pipe that the product coming out at the back end seldom bears any relationship to the point of sample. In any given clarifier, I believe that two samples are important.

The Reaction Well

This point in the clarifier contains all of the additives, the recirculating sludge and the incoming water. My preference would be to sample it from above, using some sort of sample thief; that is, a bottle with a stopper which can be pulled at predetermined depths. This sample can then be used for observation or for jar study. If the solids in the sample settle and leave a relatively clear overflow, then the clarifier is being at least adequately treated and it is likely that the outlet will also be in good shape. If this sample settles and leaves a turbid overflow, there is a likelihood that the unit is deficient in alum or coagulant. Further sampling and jar studies at this point should provide a guide to corrective action. Whatever happens, this sample should be retained. There is not a lot of point taking a sample one day without having a reference from the day before or the week before to compare. I would like to see a series of 10 – 12 samples sitting on the control lab shelf at all times, replacing only the last one when a new sample is taken. Observation of the changing characteristics of the solids/liquids mixture is probably the best control procedure one can devise, and it is certainly better than the volume/volume 10 minute sludge volume measurements which are reported in thousands of log sheets, but which do not take into account anything about the fluffiness of the sample, the size of the floc, or the clarity of the overflow.

By the same token, two underflow solids samples should be taken daily and retained for at least a week to observe changes in the underflow characteristics. Clearly, if underflow solids concentration is declining, desludge rates should be adjusted, but some comparison is necessary.

[Figure 6] The Graver Reactivator has sample points in the upper and lower draft tube which are supposed to represent recirculating sludge, and recirculating sludge diluted with incoming water. The theory is that there should be a 25 - 40% difference in the two sludge volumes. Again, this begs the question of sampling procedure and floc characteristics, and gross numbers are difficult to predict at any rate.

TROUBLE SHOOTING

So, what do we do when we walk out onto the bridge and look down into this enormous tank full of dirty water, recognizing that the filters, at this point, are backwashing continually and that the final product will be making the citizens scream as soon as they see it coming out of their taps? Well, the first thing to do is to check alum demand. Do a jar study, check turbidity levels, look back at historic alum requirements, or whatever method determines the alum demand that you use.

The second thing that you do is to check the alum feed. This does not mean to check the meter, it means check pump output, tank drawdown, pump input, but certainly check in some physical way, how much alum is being fed and calculate the physical alum feed against the requirement. If you are using polymers, do a similar check on the centre well to observe its settling characteristics. If it is settling well and leaving a clear overflow, then the clarifier will not remain upset for long.

Check sludge levels. We have found more clarifiers full of sludge and carrying over than we have ever found empty. Check the quality of the desludge material. We have also found numerous desludge lines which have become totally plugged, so that while the valve opens conscientiously, nothing appears at the outlet.

If the reaction well sample appears well coagulated, but cloudy, check turbine speed. High tip speed turbines can easily shear alum floc to the point where it becomes micro crystalline. This material cannot settle and will wind up binding the filters. In alum clarification, turbine speeds do not have to be high to provide adequate recirculation of floc.

Those of you who have high silt or sand levels may wish to consider desanding techniques in your desludge cycles. Sand or silt can create high torque without any attendant sludge blanket, simply because they settle readily, but do not recirculate at all. The best technique is to speed up the rakes (accept a short term higher torque) and increase blowdown frequency, maintaining very short desludge intervals, e.g. 30 seconds each 2 - 3 rake revolutions (at 0.1 RPM, this means 20 - 30 minutes).

Finally, whatever you do, please try to make changes one at a time and give enough time for the change to take effect. It is too often that we hear the comment, "I've tried everything, and nothing works", and we find that the operator found an upset clarifier, and proceeded to change alum dosage, polymer dosage, turbine speed, desludge rate and, for good measure, flow. Even if it did work, we'd never know which parameter was wrong.

TAPER CONTACT FLOCCULATION

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The purpose of this study is to highlight the concept of "contact flocculation". The effect of different parameters on the flocculation process is examined. A new flocculation process named "Taper Contact Flocculation" is proposed and an equation for designing flocculators of taper contact flocculation is derived.

INTRODUCTION

Flocculation is one of the important steps in water treatment. In order to define flocculation, it is important first, to define coagulation. Coagulation is the process of chemically altering collides to be able to approach each other and form larger particles while, flocculation is the physical process of bringing the coagulation particles into contact to promote floc formation.

Flocculation can be achieved perikinetically through the natural phenomena of Brownian motion or orthkinetically by inducing mixing to the water body Collision of particles due to Brownian motion is a slow process, therefore, accelerating such collision by mixing the coagulated water is advantageous.

Several configurations for flocculators have been proposed and practiced through the years. They can be classified into four groups according to Ives:

1. pedals:	revolving on horizontal or vertical axes, oscillating, vertically or pendulum-like, propeller or turbine blades;
2. baffles:	fixed in channels or tanks providing either 'over-under' or 'round the end' motion;
3. pipes	turbulent flow provides the power dissipation;
4. particles:	fixed beds (filters), fluidized beds (floc blanket, with possible solids recirculation) bubble swarms, differential settling.

The most common flocculators used world wide were of the first and second groups. Till now (to the author's knowledge), there is no fixed bed flocculator that has been designed and used in a real water treatment plant.

The purpose of this study is to present basic modifications to contact flocculation and to add a new member to the flocculator family. This flocculation method will be named "Taper Contact Flocculation" and accordingly the flocculator will be named the "Taper Contact Flocculator"

IMPORTANT FACTORS AFFECTING FLOCCULATION

Ives (9) used the term "volume flocculation" to describe the effect produced by gentle and prolonged stirring within a confined chamber of the coagulated particles in order to form flocs able to either settle or be removed directly by the filter bed. Therefore, flocculation depends on:

1 - Number of particles present,

2 - Probability of collision due to charge reduction

The probability of collision increase or decrease depends on the degree of mixing induced in the mixing vessel which is traditionally measured by the time of mixing (t) and the velocity gradient (G) as developed by Camp (3). Optimizing these parameters to obtain the best performance of the flocculation process is the target of most of the previous works.

The number of particles present depends on the water source and the coagulant dose. The control of this parameter is not the scope of this study. Therefore, optimizing the flocculation process will focus on the two parameters G and t. The optimization of G and t to obtain the best flocculation is also necessary to obtain the most economic design which means the minimum cost for an efficient flocculation tank. Talking numbers, this can be achieved with minimum and uniformly distributed G value (i.e less power input), and minimum t (i.e. minimum tank volume).

BACKGROUND

Camp (2) suggested 8,000 - 10,000 as an optimum G.t number for alum flocculation, and 30,000-50,000 for ferric chloride; the difference was due to the strength of ferric hydroxide flocs.

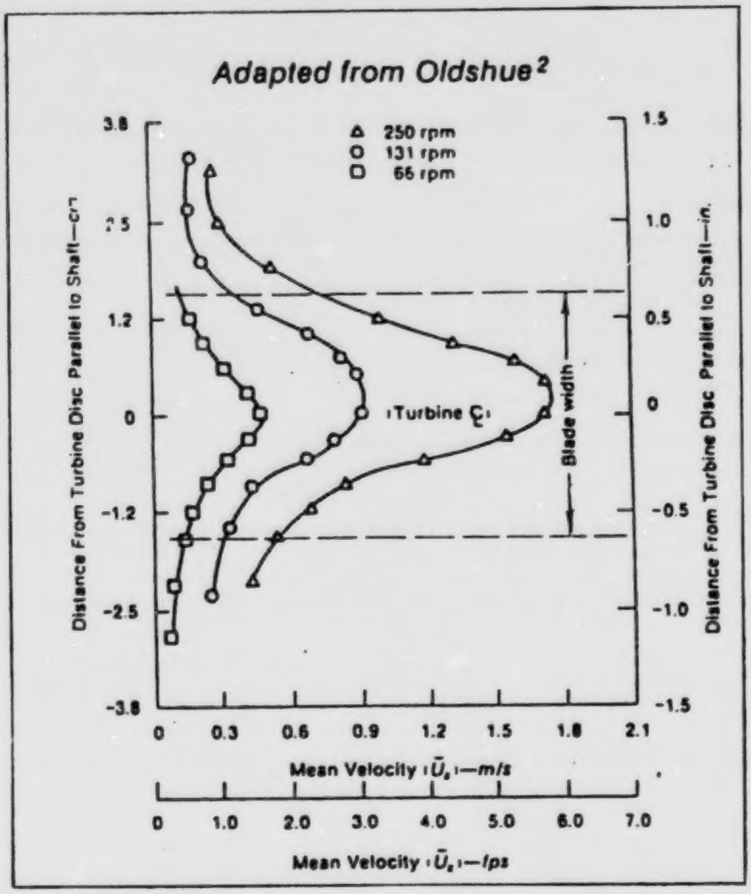
Most text books recommend Gt values of 10,000 - 100,000 for design. Monscvitze et.al. (15) proposed 30 min. flocculation time and Gt from 25,000 to 50,000 for direct filtration process. McCormick and King (14) observed that when cat floc polymer was used flocculation for 28 min. and G from 20 to 63 s^{-1} (Gt = 33,000 - 106,000) improved the effluent turbidity marginally. Andrew - Villegas and Letterman proposed 20 to 30 min. flocculation time and an optimum G value in the range of 20 to 50 s^{-1} (Gt = 24,000 - 90 000). Hutchison (7), in a direct filtration pilot scale investigation found that the best runs were obtained with G of 20 s^{-1} and 5 = 4.5 min. (Gt = 5,400). The fluctuation of the G and t values, and accordingly the Gt value, is too wide and confusing.

As mentioned previously, the best flocculation is the one that rapidly forms large separable flocs, leaving no residual primary particles or small aggregates. The G value reported should not be more than 100 s^{-1} otherwise floc break-up will predominate. This G value is the average value which occurs in a basin. In reality the velocity gradient at any instant through the vessel or chamber in use for flocculation will be greatest at the solid boundaries of the padels or other devices used to introduce the mixing motion and is reduced farthest from the point of introduction of motion, Figure 1, adapted from Oldshue (16).

This may be the reason for widening the flocs size distribution and the increase of the required time for flocculation to take place. Some areas of the tank have a higher G value than the assigned one, while, other parts have lower G values. This means that the power is not uniformly dissipated inside the tank and floc formation and damage recur many times.

FIGURE 1

Water velocity at various positions above and below a 15 cm (6 in.) diameter turbine with six flat blades

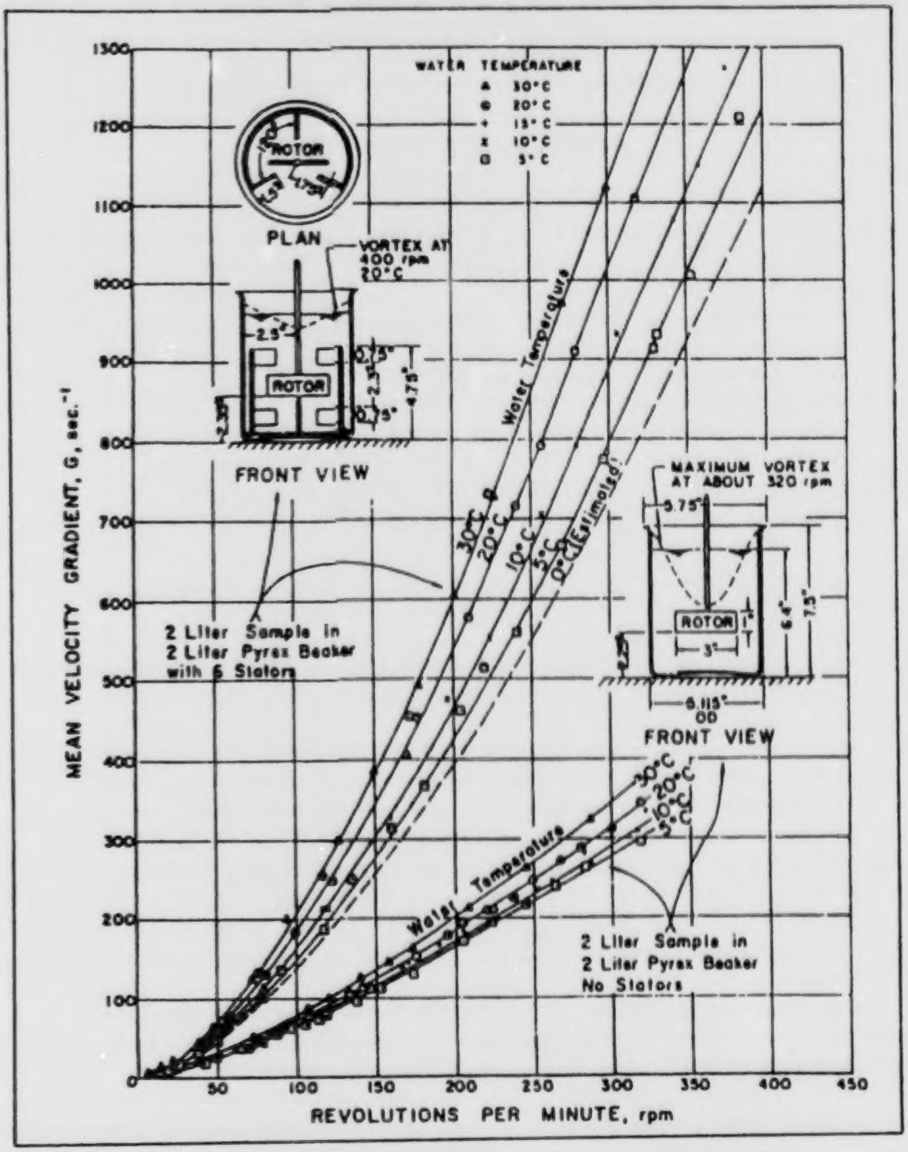


Camp [3] showed that adding stators for a 2 liter beaker increased the G value to 3-fold more than a beaker without stators for the same power input. Also, the presence of stators increases the power dissipation 10 times over that without stators, Figure 2.

The author believes that if the G value will be uniformly distributed in the vessel, the time required for flocculation will be much shorter than that stated in the literature, and the flocs will have a narrow size distribution.

FIGURE 2

Velocity gradient calibration curves for water



CONTACT FLOCCULATION

From the above discussion, we can conclude that vessel work on contact flocculation or particle flocculation should provide a uniform power dissipation and almost one G value. The water flocculation should provide a uniform power dissipation and almost one G value. The water falling from the top to the bottom by gravity in a column containing granular particles provides the power added to the water body, the head loss across the media is the power dissipated into the water body, and the particles that face such water are the stators in the beaker. Having the particles with one diameter and arranged uniformly will dissipate the input power of the falling water uniformly and flocs formation will occur with a narrow size distribution.

One important draw back is that, at the time the tank functions as a flocculator, it works as a filter. Accordingly, the pore size of the media will decrease with time and more head loss will build up. This condition of filter action will:

1. increase the power input
2. increase the G value to the limit which may cause more flocs damage than flocs formation.

Orthkinetic contact flocculation has been mentioned and studied as an observed phenomenon occurring across the filter media by several researchers [8,11,15].

Ives mentioned that in case of fixed bed granular filters, flocculation occurs when power dissipate as laminar flow through granular media. He used Kozony-Carman equation for head loss through a fixed bed.

$$\frac{H}{L} = \frac{5\,uv(1-e)^2}{p\,g\,e^3}\left[\frac{6}{D}\right]^2 \qquad (1)$$

where L is the bed depth, u is water viscosity, v is the superficial velocity above the bed, e is the porosity of the fixed bed and D is the fixed bed particles diameter.

Using equation 1 and the relation between the power dissipation and the head loss, he found that

$$G = \frac{13.4\,v\,(1-e)}{e^2\,D} \qquad (2)$$

where v is the filtration rate (m/hr). Ives stated that "In studying flocculation in such fixed beds it is difficult to follow the kinetics of flocculation, or even to study the reduction in the numbers of primary particles, due to the removal of so many flocs by deposition on the fixed bed grains (filter effect)".

Ives presented an example, where he assumed a fixed bed having grain size D = 0.5 mm, porosity of 0.4 and thickness L = 600 mm. The G value, on applying his equation, was 140 s^{-1}. In this example which presents normal backed media used for filtration process, the G value is high and large flocs do not form. The filter pores will clog rapidly causing G value to increase and, as mentioned previously, may reach the point where no flocculation takes place at all.

Taper Contact Flocculation

Bearing in mind the problems associated with the use of contact as well as mechanical flocculation mentioned in this paper, the author proposes the Taper Contact Flocculation (TCF). The TCF, suggested herein, comprises the use of granular media which consist of 3 or more layers of almost spherical particles of any kinds of materials. This material should have a high specific gravity and should not react with water or the chemicals used in the process such as glass beads or well rounded gravel. The top layer should have a particle diameter not less than 3 mm, the second layer should have a larger particle diameter (for example 6 mm), and so on, for the third and fourth layers. The surface loading on the flocculator should be as high as 15 m/hr or may be higher, according to the required G value.

The reasons for these characteristics of the suggested flocculators are:

1. The large size media is essential for increasing the pore size and reducing the specific surface area of the media in order to reduce the effect of filtration significantly.
2. The configuration of the suggested layers is important for allowing the growing up of flocs inside the flocculator. Also, in case of the top layer being partially clogged by suspended solids deposition, the remaining layers (having larger particle diameters) will flocculate the escaping suspended solids.
3. The high surface loading is advantageous for reducing the filtration actions and is essential for obtaining the designed G value for the large media sizes suggested for the process.

G Value Calculation

With such high surface loading and the large media sizes proposed for the TCF, the Kozony-Carman equation is no longer adequate. This equation is used for laminar flow conditions, in which the Reynolds number is below 6 as reported by Camp (2).

$$Re = \frac{D\ v\ p_w}{u}$$

where D = the grain diameter, and
v = the superficial velocity above the bed.

The suggested flow may be in the transitional flow regime. Therefore, the Ergun equation (3) should be used because it is adequate for the full range of laminar, transitional, and turbulent flow through packed beds.

$$\frac{H}{L} = 150\ \frac{p\ u}{g}\ \frac{(1-e)^2}{e^3}\ [\frac{1}{D}]^2\ v + 1.75\ \frac{(1-e)}{e^3}\ \frac{1}{D}\ \frac{v^2}{2g} \tag{3}$$

When packing spherical large particles, the porosity could have a minimum and a maximum value depending on the arrangement of the particles. As shown in Figure 3, the porosity for a is 0 476 and that for b is 0.26. Considering an average porosity of 0.368 to be the porosity of a randomly arranged bed, equation (4) will be:

$$\frac{H}{L} = 1202\ \frac{p\ u}{g}\ [\frac{1}{D}]^2\ v + 22\ 2\ [\frac{1}{D}]^2\ \frac{v^2}{2g} \tag{4}$$

Knowing that:

$$P = p\ g\ Q\ H \tag{5}$$

where P is the power dissipated

and $G = [P/Vu]^{1/2}$ [6]

where V = the layer volume = e A L (A is the layer surface area).

Then $G = \left[\frac{pg}{ue}\ \frac{Q}{A}\ \frac{H}{L}\right]^{1/2}$ [7]

Substituting, the values of H/L from equation (4) and Q/A by v, in equation (7), G will equal to:

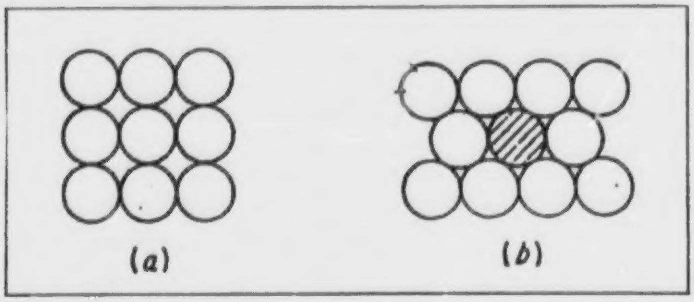
$$G = 7.77\ v/D\ [54.4 + R_e]^{1/2} \qquad [8]$$

Equation (8) can be used to determine either one of the three parameters G, v, and D. Knowing or proposing two of these parameters, the third one can be obtained.

An example for using equation (8) can be illustrated as follows:

> Assume a flocculator consisting of three granular layers with G values of 100, 70 and 40 s^{-1} and a surface loading velocity of 15 m/h, the required diameters by trial and error will be 5, 10, and 28 mm. respectively.

FIGURE 3


(a) (b)

Recommended Future Works

The surface area of the TCF can be obtained by dividing the discharge over the superficial velocity.

The depths of the various layers are the missing part of this study. These depths can be determined if the t or the Gt value is known. The only way to determine the required retention time of the TCF is to run an experimental work that comprises the use of flocculation-filtration or flocculation-sedimentation process where depths of the layers will be monitored to obtain the best flocculation-filtration or sedimentation process. Right now, experimental works are conducted in El Mansoura University, Sanitary Engineering Lab to determine the optimum value for the Camp number for the TCF.

Additional experimental work is required for comparing the efficiency of the mechanical flocculators and the TCF. Also, the best configuration for the TCF, including the inlet and outlet to the flocculator media and the method of backwashing such media when it is necessary, should be studied carefully.

Summary and Conclusion

The uneven dissipation of power input in the water body subjected to flocculation is the reason for the wide range of flocs size distribution and the long retention period required for flocs formation. The reported Gt values in the literature ranging from 5400 to 205000 are too wide. The main reason for a wide range is that different researchers used different configurations for their flocculation processes.

Fixed bed contact flocculation was studied as a phenomenon that occurs in the filter media during the filtration process, but never used in practice as a flocculation process. Taper contact flocculation is a modification of the contact flocculation. This method comprises the use of layers with larger particles diameters than that found in the ordinary filter media. These layers are stacked in ascending order i.e. the small diameter layer in the top followed by the larger one..etc. The flocculator media should be consisted of spherically uniform size particles. The surface loading rate used for flocculation is much higher than the filtration rate assigned for filters.

The characteristics proposed for the TCF are important to reduce the filtration effect as well as to allow the flocs to grow up across the flocculator.

The depths of the layers will be determined experimentally in the near future. Also, experimental work is needed for establishing the TCF tank configuration, i.e., the inlet and outlet arrangement, and the required backwash (when and how?).

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DEVELOPMENT OF A TRANSPORTABLE FACILITY FOR THE ON-SITE TESTING OF ALTERNATIVE TECHNOLOGIES IN DRINKING WATER TREATMENT

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INTRODUCTION

As basic water management policy, the Alberta Government recognizes the importance of water for human consumption, for food production, for industrial use and for other uses in that order. This order of preference is embodied in the Alberta Water Resources Act. The priority of potable water as the most essential of all uses is therefore fundamentally recognized within the legislation of the Province of Alberta. On the basis of this powerful mandate, the Potable Water Technology Research Program was developed at the Alberta Environmental Centre (AEC). It is intended to find solutions pertinent to problems in potable water treatment and distribution experience within this Province.

Water resources are found in relative abundance in most areas of the Province and consist of surface water and groundwater supplies. The quality of these source waters varies greatly throughout the Province. In the Rocky Mountains and foothills, cool streams of high clarity appear to require minimum treatment options. Yet, it is in water originating under these circumstances that the threat of waterborne giardiasis is most pronounced, the cysts passing unchallenged through treatment systems designed for minimal treatment of high quality waters. The identification of giardiasis as a foremost public health concern has lead to a concerted effort to develop viable detection and treatment options for the causative organisms.

On the prairies, in steams swollen with surface run-off during high precipitation periods, turbidity, colour, taste and odour problems are recurring concerns. In areas of limited supply, water availability is restricted to high flow conditions and impoundment is necessary. Problems associated with quiescent waters such as algal growth are common.

Also, the problem of high haloform formation potential is a typical Prairie phenomenon.

Even though industrial activity is limited in the province, specific discharges do present concerns. In northern and west-central regions, the activities of pulp and paper mills result in the need to assess the impact of discharges from these types of operations on the subsequent treatability of downstream surface waters for the purpose of producing a potable quality product. The study of trace organics and their response to various treatment options is pertinent to these circumstances. Unique to northern Alberta, the tar sands plants and their tailings ponds present potential discharge concerns similar to those imposed by the pulp mills. Effluent discharge licenses can only be considered in concert with an understanding of the effects on downstream drinking water supplies.

When surface supplies are unavailable or unsuitable, groundwater becomes an important source. Unfortunately, many areas have groundwater of unacceptable quality, with high levels of total dissolved solids and iron. Also, the presence of H_2S is not an uncommon occurrence. These are some of the more traditional treatment challenges. With the current recognition of the potential for groundwater contamination by agricultural, municipal and industrial practices or by accident, research into groundwater treatment has become important and timely.

RESEARCH FACILITY REQUIREMENTS

In support of the search program formulated to address these problems, the development of test facilities was required. This development incorporated in-house systems at the AEC and field facilities.

Due to the site specific nature of many water problems and the alteration of water characteristics with sampling, transport and storage, a heavy emphasis has been placed on the development of field or on-site research capability. The advantage of achieving this result with portable systems is the capability for the rapid development of specific field pilot plants by the modification of a core unit while conveniently located at the AEC. When ready, it is simply moved to the study site of interest. At present, this philosophy has led to the construction and development of two units: the first a demonstration-scale, 200 Lpm unit using current North American technology and the second, a smaller 4 - 24 Lpm unit, containing alternative treatment technologies, namely ozonation and upflow clarification, as well as conventional filtration, clarification and chemical treatment options including chlorination and chloramination. It is the design and utilization of this second unit that will be dealt with in the following sections.

FACILITY DESIGN

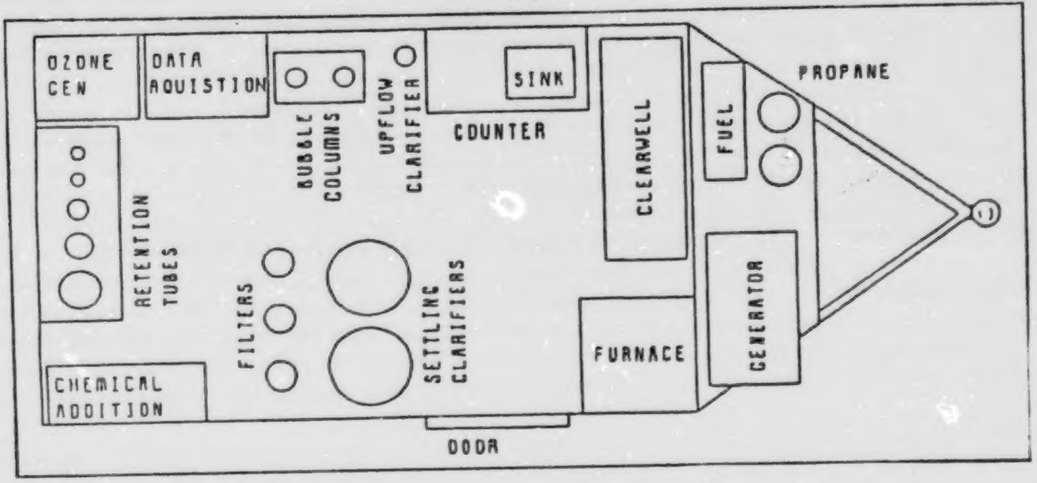
General Requirements

Based on the design intent of full field operation capability, the unit was equipped with self-contained power generation and a propane heating system within a fully weatherproof housing. The diesel generator supplies 20 KW (26 8 HP) to the system, sufficient to operate equipment such as delivery pumps, an ozone generator, chemical feed units and a variety of monitoring equipment Alternatively, the trailer can also be hooked up to an external power source.

For ease of transport (no travel restrictions) and use, the system was designed at a nominal capacity of 8 Lpm (hydraulic capability of 4 to 32 Lpm) This permitted the use of threaded pvc piping, allowing for the easy alteration of the system. Alternative flow schemes can also be achieved by various valving options. A fiberglass based floor with a skirt running one foot up all walls provides containment of any spillage or upset and protects the undercarriage from undesirable seepage.

The physical plant (trailer) is 3 m (10 ft) wide by 4.9 m (16 ft) long with an internal ceiling height of 2.4 m (8 ft) and a total transportable (no water loading) weight of 4100 kg (9000 lb). The internal floor layout is shown in Figure 1.

FIGURE 1
Plant Layout



Data Acquisition and Control System

Part of the operation of experiments in the trailer is the collection of data for water quality and other experimental parameters. One method of accomplishing this is by labour intensive sampling and lab analysis. The concept developed for the trailer is far removed from this as one of hands off computer collection and analysis of data. Continuous monitoring instrumentation connected to a computer which can store data and later retrieve it for analysis and final presentation provides this capability. Labour requirements are reduced to calibrating the instrumentation, initializing the data collection, and providing input into the analysis of raw data.

An additional part of the concept for this computerized system was its ability to provide the potential of automated control of the treatment plant.

In addition to meeting the conceptual requirements the data acquisition and control system was chosen for its low cost, compatibility with IBM or similar computers within the Research Centre, and the flexibility of using the system in many operating environments. Instruments were selected on the basis of their ability to provide continuous monitoring a reasonable response time, adequate sensitivity to measure water quality parameters, ruggedness for field operations, and low maintenance requirements (ie. no filling solutions).

The data Acquisition System presently consists of the following components:

- One IBM compatible computer
- One METRABYTE Dash - 16 data acquisition and control card
- One redox meter and probe
- One free Chlorine analyzer
- One pH analyzer
- One Conductivity analyzer
- One Flowmeter with signal conditioner
- Two RTD temperature sensors
- One Labtech Notebook software package

The instruments to monitor water quality have local readouts as well as a 4 to 20 mA current output. The current output is routed through a 250 ohm resistor at the data acquisition card, providing a 1 to 5 volt signal. The Dash - 16 data acquisition card then provides real time analogue to digital conversion of this signal corresponding to the water quality parameter being measured.

The Dash - 16 has the capability of reading eight different signals in this configuration. It also has the capability of sensing 4 digital inputs, changing the status of a 4 digital outputs, and controlling 2 digital to analogue outputs.

Control of the Dash - 16 is provided by a commercial software package (Labtech Notebook). This software provides an interface to set functions of the Dash - 16 data acquisition card such as sampling rates, signal input, signal output, displays and storage. The package also has the capability of real time display in converted units of the incoming signal as well as storing data on disk in files compatible with LOTUS Prn files. Curve Fit and Fourier transform functions also are available which allows data analysis in conjunction with LOTUS.

The sensors on the Redox, Chlorine, pH and Conductivity analyzers are installed in flow cells. Through a plumbed in sampling system a water sample can be run through the flow cells from one of eleven different sampling sites in the system.

A flow reading is taken at the outlet of the clarifiers and the end of the treatment train. Rotameters are also provided at the inlets of the clarifiers and filters to allow for balancing of flow.

Temperature is measured at the inlet and outlet of the plant.

The plumbing on the sampling system is designed so that the flow from any sampling point reaches the flow cells in less than ten seconds. However, the volumes of the flow cells as compared to the sample flow, which must be small compared to the total flow through the plant, is such that the response time of the water quality analyzers is in the order of two to three minutes.

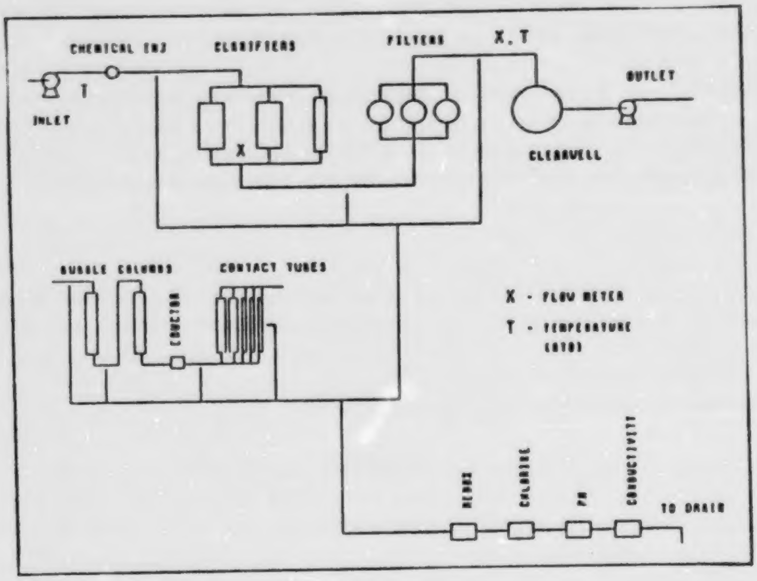
Figure 2 illustrates the sampling and data gathering system in relation to the process operation of the trailer

Ozone Generation

The trailer has been supplied with an ozone generation unit rated at 12 gm/hr (air feed) or 17 gm/hr (enriched O_2 feed) maximum. Oxygen enrichment is achieved by passing air supplied by an oil-free air compressor through a water trap and then through an adsorber column (containing molecular sieve), resulting in 65% to 68% - O_2 content at 25 CFH. Consequently, the unit can supply a dose rate of 2.5 mg/L to 35.4 mg/L at the normal hydraulic flow rate of 8 Lpm.

FIGURE 2

Sampling System


CHEMICAL INJ
CLARIFIERS
FILTERS
X, T
OUTLET
T
INLET
X
CLEARWELL
BUBBLE COLUMNS
CONTACT TUBES
EDUCTOR
X - FLOW METER
T - TEMPERATURE (RTD)
REDOX
CHLORINE
PH
CONDUCTIVITY
TO DRAIN

Ozone Reactor

Independent of the means of ozone generation is the facility required for providing the contact of the ozone, produced in the gas phase, with the water undergoing treatment. Variables impacting on this process include temperature, pressure, pH organic and inorganic content of the water and ozone stability.

Units by which this process can be accomplished include bubble columns, in-line educators, in-line porous stone/static mixers, agitated tank or eductor recycle, packed columns and spray chambers. These different techniques all have various inherent advantages and disadvantages based on dosage versatility, space requirement, efficiency, capital cost, ease of scale-up, equipment maintenance, monitoring convenience, availability of design criteria information, construction material requirements and local industrial support.

For the present system two units were chosen. Firstly, an eductor, based on its low cost, simplicity and low space requirements. Secondly, a two-column bubble contactor, based on the following reasons:

- Has available a large body of information regarding design and utilization in the literature.
- Allows for primary and secondary dosing in the two step addition process, superior to single dosing alone.
- Allows for a 9 minute contact time at a flow rate of 9 Lpm, 200% of the recommended kill time for most organisms.

- Allows for changing of flow patterns to accommodate parallel or series flow in cocurrent or concurrent types of reaction schemes.

- Allows for variability of contact time.

- Allows visual inspection via clear plastic contact column.

- Column heights could be tailored to interior height limit of trailer.

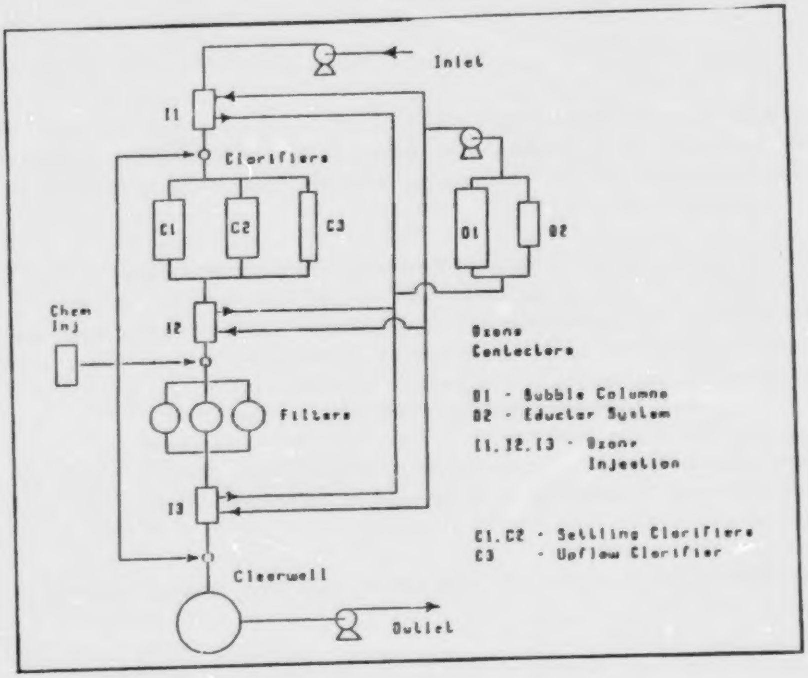
- Can be adapted later to make use of an extended column, mounted on top of trailer, for the purpose of preozonation with the offgas from the lower column.

TREATMENT PROCESS

This pilot plant unit is intended for the application, testing and evaluation of alternative treatment processes not familiar to the province, either seperately or in conjunction with conventional treatment practices. The goal is to identify optimized treatment capability based on currently available technology and consequently improve upon the quality, quantity and/or cost of water presently available to communities within Alberta.

FIGURE 3

Process Flow


Inlet
I1
Clarifiers
C1
C2
C3
O1
O2
Chem Inj
I2
Ozone Contactors
O1 - Bubble Columns
O2 - Eductor System
I1, I2, I3 - Ozone Injection
Filters
I3
C1, C2 - Settling Clarifiers
C3 - Upflow Clarifier
Clearwell
Outlet

Processes identified within this context include ozonation and upflow clarification. Figure 3 illustrates the current configuration of the pilot plant to include these unit processes in the treatment scheme in a variety of operational scenarios.

The potential role of ozone in drinking water treatment in North America and elsewhere was recently extensively discussed at the Second International Conference on the Role of Ozone in Water and Wastewater Treatment in Edmonton, Alberta on April 28 - 29, 1987. However, local experience with ozone is limited. Some unsuccessful attempts using ozone form part of this background. Consequently, a sense of apprehension and concern encompasses the concept of ozone utilization. Effective demonstration of ozone on a provincial level via the current pilot system would help assuage these concerns and assist in the effective implementation of this technology.

Upflow clarification is starting to be introduced to Alberta. To place this process in proper perspective to other treatment practices with respect to Alberta treatment requirements, this technology has also been included in the mobile pilot plant for evaluation under Alberta water conditions.

CONCLUSION

The two primary areas that are presently the focus for studies involving this mobile pilot system include the study of treatment options to ensure the microbiological quality of potable water and the study of water treatment for the purposes of removing colour, taste, and odour. The former is a logical extension of our current study of conventional filtration and disinfection processes and their difficulty in ensuring adequate protection from organisms such as Giardia Lamblia. The latter is due to the prevalence of highly coloured surface waters in Alberta and elsewhere on the Prairies and the current concerns raised with respect to the application of large doses of chlorine in treating these waters. Consequently, ozone, possibly in combination with clarification procedures such as upflow clarification, provide treatment alternatives which, based on experiences elsewhere, provide excellent potential for improved treatment of Alberta waters for the removal of colour, taste and odour.

REMOVAL OF SILT, SLUDGE, AND SEDIMENTATION FROM SEWAGE LAGOONS, SETTLING PONDS, AND NATURAL WATERWAYS

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Conventional methods of silt, sludge and sediment removal has generally consisted of:

1. Using a dragline and working blind.
2. Dewatering the problem area, allowing the area to dry then moving in with heavy equipment and removing the material and transporting it to a disposal site.

PROBLEMS

Method #1: Dragline is working blind and has very little if any control over excavating depth or following pre-set bottom contours. In sewage lagoons, this may result in deep ruts and breaking of the bottom seal of the lagoon, creating numerous environmental problems.

Method #2: Dewatering lagoons means a lengthy shut down of the system with a resulting overloading of remaining lagoon cells.

SOLUTION

Dominion Divers Ltd. has developed the SUB CAT, a submersible sludge and silt removal machine that does not require system shut down and eliminates damaging the bottom seal of the lagoon.

SUB CAT is a light weight self propelled hydraulically driven crawler running on two 32" wide tracks resulting in a ground pressure of less than .5 lb per square inch.
Mounted on the end of the SUB CAT are two opposing hydraulically driven augers that break up the silt and sludge and force feed this material into the suction of a 6" dredge pump. Feed augers are contained in a silt shield that may be raised or lowered resulting in infinite control over excavating depth and the ability to follow the natural contours of the lagoon or Pond.

Power unit and pump are mounted on two inflatable pontoons that float on the surface and are towed by the SUB CAT, this method permits the SUB CAT to be completely independent of any surface wind conditions and allows maneuverability even in aerated lagoons.

Dredged material may be pumped a distance of 500 ft into a pre-determined spill area where it then may be spread out over fields or allowed to dry then hauled away by trucks.

SPECIFICATIONS

Weight:	8,000 lbs.	**Track Ground Pressure:**	Less than .5 PSI
Pumping Distance:	500 ft	**Material Removal Rate:**	20 - 70 yards per hour depending on material density and distances to be pumped

ACCOMPLISHMENTS TODAY IN CHEMICAL FREE WATER TREATMENT FOR A BETTER ENVIRONMENT TOMORROW

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INTRODUCTION

Many times we have heard the statements like: "Our national resources are depleting", "The quality of our water reserves are deteriorating", "We are polluting our water ways with harmful chemicals". "Do not swim — The water is dangerous to health". We cannot solve all these problems overnight, since especially in the water and wastewater industry we are not in control of the industries that use water in their processes, which add harmful chemicals to the process water.

What we are dealing with specifically in our industry is the purification of water for potable use or cleaning up our wastewaters after mainly domestic use.

This paper will address only the potable water treatment aspects of the method available today.

TOXINS AND CHEMICALS

Even if potable water treatment is responsible for only a portion of the problem of the introduction of toxins and chemicals in water and treatment waste we cannot ignore this, but rather must work together to minimize or eliminate the use of any chemicals where ever possible.

Time after time we are still using chemicals. Sodium Hypochloride, Aluminum Sulphate, and Potassium Permanganate just to name a few commonly used ones. On every water treatment plant or upgrading of existing facility we read in the specifications: chemical feed equipment shall be this or that, feed rates for chemicals A or B. Special rooms to store chemicals or even the chemical feed equipment. Feed pumps are getting more and more sophisticated and the instrumentation and monitoring equipment more and more costly. – And Why? Do we not have available any process that is simple, that is inexpensive and that uses less or even no chemicals? The answer is yes.

TREATMENT

To treat approximately 75% of the raw waters in Western Canada, we do not require any chemicals for water purification. The most common problems in our water such as iron, hydrogen sulphide, and manganese we can now remove without harsh chemicals. Even tannin and organic contaminants are removable without the use of chemicals.

Our first attack on chemical usage is on well water in which we commonly find dissolved iron, H_2S, and manganese. These are matters which can be removed from solution by simple oxidation followed by filtration. A rusting bar and a burning branch are both examples of oxidation. In one case the product is iron oxide or rust, in the other case the products are ashes, various gases and water vapour. Water that is proposed for both domestic and industrial uses can be afflicted with the problems of iron and organics as in the examples of the rusting bar and the burning branch. They can be treated by the process of oxidation for the elimination of these problems.

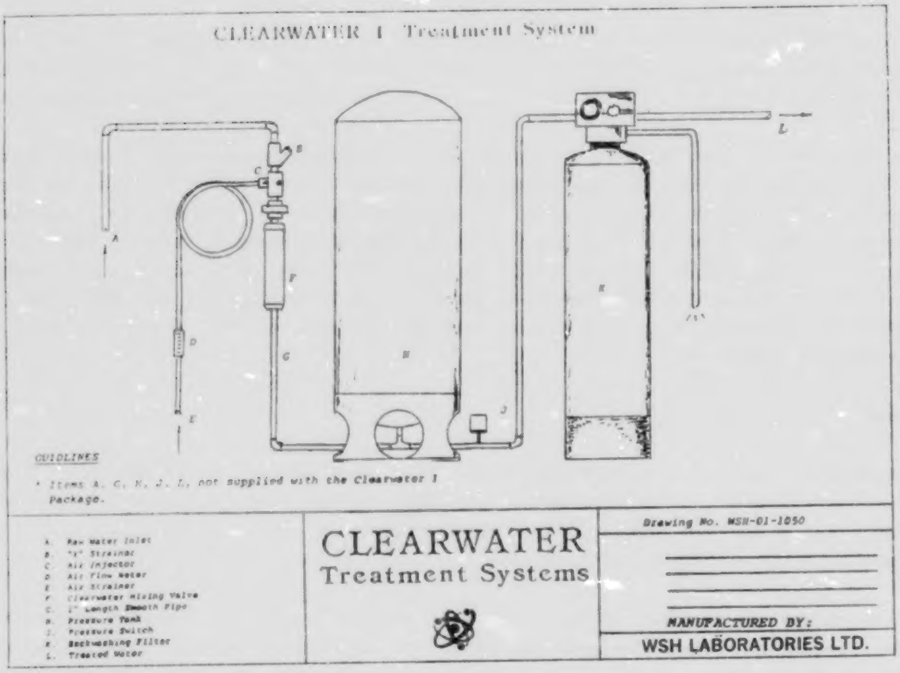
The traditional way of treating these problems is to add chlorine or potassium permanganate to change these substances into a form which can be filtered out of the water with physical filtration. The use of these chemicals, while solving the iron and to some extent the organic problems, have certain drawbacks and indeed introduce some problems of their own:

- the chemicals themselves are toxic
- they do not work satisfactorily in every case
- they require frequent costly maintenance
- they produce byproducts not appreciated in potable water
- they produce a smell or taste in the water that is repugnant to many people.

Our Non-Chemical method (figure 1) for iron removal uses oxygen itself to change the unpleasant characteristics into a form where they are readily filtered out. The oxygen is not at all in short demand as it is available and usable from the air we breathe. Using the air and its oxygen content is nothing new.

FIGURE 1

Clearwater Treatment System


CLEARWATER I Treatment System
A
B
C
D
E
F
G
H
J
K
L
GUIDLINES
* Items A, G, H, J, L, not supplied with the Clearwater I Package.
A. Raw Water Inlet
B. "Y" Strainer
C. Air Injector
D. Air Flow Meter
E. Air Strainer
F. Clearwater Mixing Valve
G. 1" Length Smooth Pipe
H. Pressure Tank
J. Pressure Switch
K. Backwashing Filter
L. Treated Water
CLEARWATER
Treatment Systems
Drawing No. WSH-01-1050
MANUFACTURED BY:
WSH LABORATORIES LTD.

In fact aeration has been used for a long time. The key to the success of the Non-Chemical treatment systems however, is that we have taken this process a large step further. The benefits of using oxygen consequently are:

- elimination of the use of poisonous chemicals,
- eliminates most tastes and odors from waters,
- requires very simple infrequent service, and
- does not produce harmful byproducts as found with chlorine and potassium permangenate.

In many applications it is necessary to dissolve gasses in a stream of liquid, eg: in sewage treatment, it is desirable to dissolve oxygen in a flow of wastewater. Similarly, the pulp and paper process involves at one stage, the dissolving of oxygen in a stream of liquid. In the wine making process, it involves a dissolution of nitrogen in a stream of wine. Up to the present, this method of dissolving gasses in a liquid has involved simply injecting gas into a stream of liquid, pumping the gas at atmospheric pressure. The rate of dissolution of gas in the liquid is thereby much lower than would be the case if the liquid were under a higher pressure. The present invention utilizes the pressure created by centrifugal force to increase the rate of dissolution of the gas in the liquid.

CLEARWATER MIXER VALVE

Here is the key to successful results in our advanced treatment method. Under ideal condition, it is possible to retain dissolved oxygen levels in water at a maximum of 8 PPM. This level would be satisfactory in some cases. However, the ideal conditions do not exist for some reason or another when we need and want it. The question therefore is how to dissolve more oxygen for use in the oxidation process. We now have a method by which it is possible to dissolve up to 32 PPM of oxygen for at least a period of time when the oxygen is required to do its oxidizing job. This is 4 times the level we only can hope to achieve under ideal conditions. This requires basically only energy and/or pressure which we generate by the flowing stream of water in the Clearwater Mixer valve. Since this device is the key component in our system concept, it is worth describing in detail. (Figure 2)

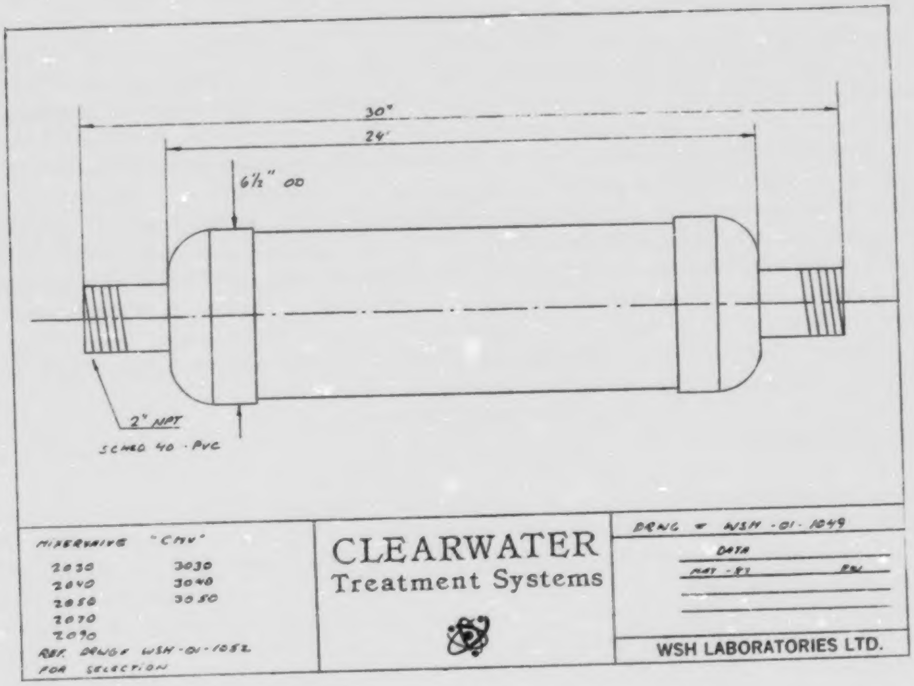
The Clearwater Mixer Valve is a unique revolutionary inline static mixing device which operates in the centrifugal force technology. Through an internal spiral design, a violent centrifugal force is applied to the flowing media which builds up pressure on impact against the walls of the mixer valve. You may call it a pressure reactor or a centrifugal contactor. This increased pressure affords an increase in dissolved oxygen when used with air injection and at the same time provides a very intimate mix. When used with injected chemicals the mixer valve has been found to speed up the reaction time with chemicals to values of 20 - 50 times faster than accomplished with the more traditional static mixers.

The sacrifice within the system for highly effective mixing is a pressure loss across the mixer valve. This mixing process could not be carried out without the use of energy which therefore is robbed from the system in the form of a headloss. The normal headloss across the mixer valve is 5 - 16 PSI, with 10 - 12 PSI as the recommended happy medium. It is therefore extremely important that correct selection is calculated prior to installation. Always know the system flowrate and pumping capacity. It is also recommended that accurate pressure gauges are installed in the system to monitor at various points:

- Raw Water Supply
- Post Mixer Valve
- Post Filter

FIGURE 2

Clearwater Mixer Valve


30"
24"
6½" OD
2" NPT
SCHED 40 · PVC
MIXERVALVE "CMV"
2030
2040
2050
2070
2090
3030
3040
3050
REF. DRNG# WSH-01-1052
FOR SELECTION
CLEARWATER
Treatment Systems
DRNG # WSH-01-1049
DATA
MAY-87
WSH LABORATORIES LTD.

Through new technology a wide range of mixer valves are available. 1/2", 3/4" and 1", are suitable at flows in the range of 4 - 30 USGPM (Figure 3). The 2" mixer valve (Series 2000 and the 3000) are designed for commercial and industrial applications on flows up to 80 USGPM. Although identical in appearance the internals of each model is carefully designed for the flow rates shown on a selection chart with the criteria of 5 PSI loss on low flow, up to 16 PSI at highest flow.

With results today from hundreds of installations, the CMV mixer valve promises simplification and cost saving on all new water treatment applications.

FIGURE 3

CMV Selection Chart

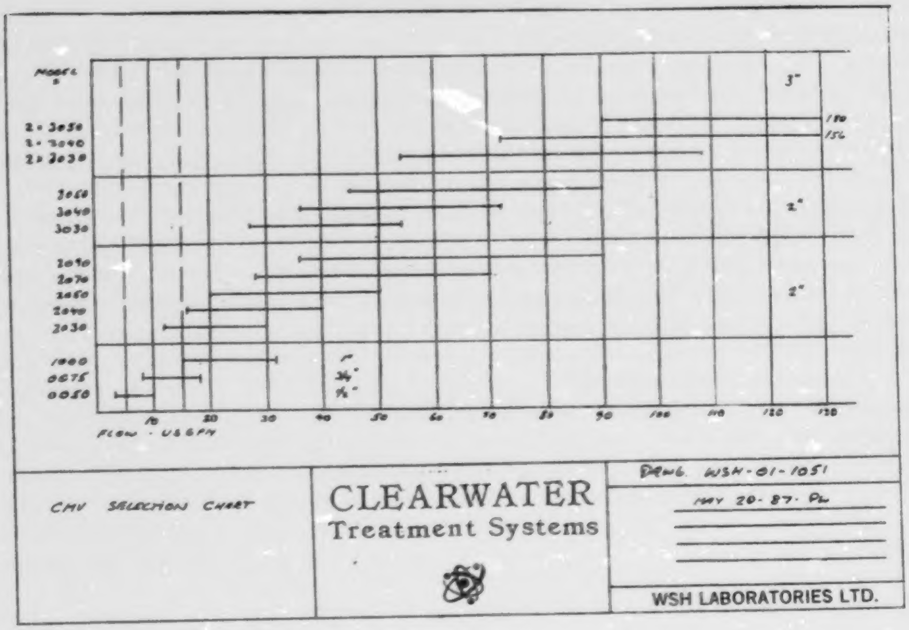
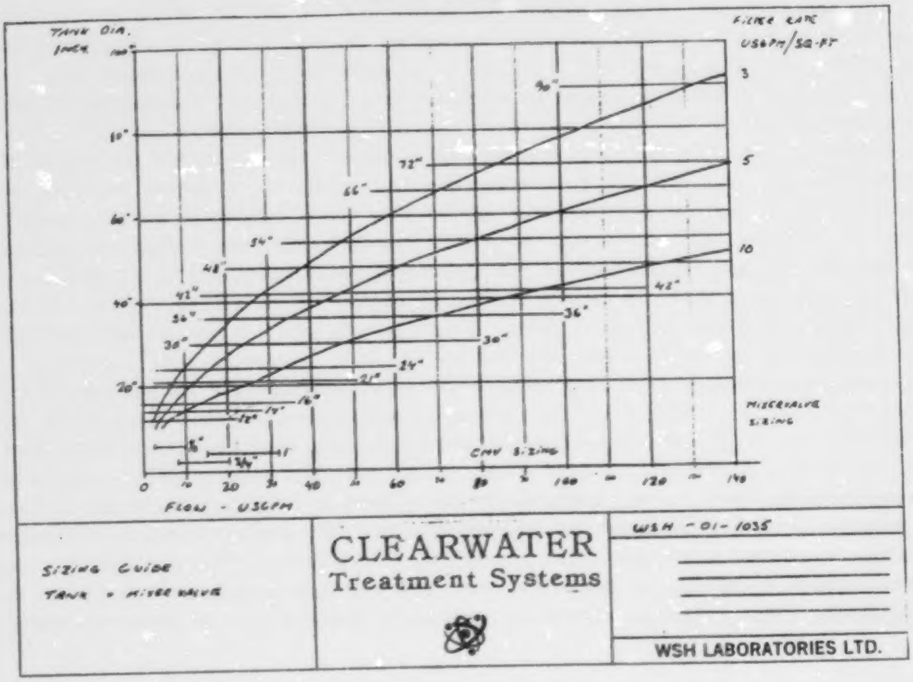


FIGURE 4

Sizing Guide
Tank and Mixer Valve



INLINE STATIC MIXER

What use to be referred to as an inline static mixer, is not to be confused with the Clearwater mixer valve. Even though it is a product we manufacture and sell as part of our business volume, this product is to be treated with respect, care and caution. The mixer valve is very powerful and it robs not just a pound or two out of the system, but 10 PSID headloss mostly. In most well waters the dissolved iron has reacted with oxygen and is completely oxidized when the mix leaves the outlet of the mixer valve. In fact the reaction is so violent that we need a few feet of straight smooth pipe afterwards to settle the turbulence. This oxidizing method has found very little limit in its performance and to date we have found water with iron levels up to 35 PPM which we have successfully treated down to less than 0.3 PPM. Manganese levels as well have been reduced successfully from 2 PPM to 0.02 PPM. Even H_2S which normally is difficult to deal with has been reduced from 3 PPM to below laboratory detection. However, no filter media can remove H_2S as long as it is dissolved, but it takes powerful oxidation to bring H_2S out of solution after which a simple sediment filter can pick up the oxidized particles. Simple sediment filtration is all that is required after a proper oxidation has taken place. This has proven to be useful when the oxidizing assembly is adapted onto an existing chemical filter as a modification kit. Quite often we have improved the performance of a regular manganese greensand filter after the changeover to the air system. No chemicals added, no mess, no toxins and no maintenance. Only good old fashioned mother natures air. If it is good enough for us to breath, it is certainly is good enough to use in our water treatment.

BIRM FILTERS

When selecting a filter media for water filtration it must be done with proper thought as well. We use almost exclusively BIRM in all our systems, including domestic, commercial and municipal systems.

Birm is a granular filter media commonly used for the removal of iron, and/or manganese from well water supplies. Birm is one of the most efficient and economical methods of removing dissolved iron and manganese compounds from raw water supplies and may be used in either gravity fed or pressurized water treatment systems depending on the application and physical design parameters. Birm acts as an insoluble catalyst to enhance the reaction between dissolved oxygen and the iron compounds found in many water supplies. Ground water supplies in many cases contain various amounts of calcium, magnesium, iron, manganese, sodium and silica. In most common ground water the dissolved iron is usually in the ferrous bi-carbonate state due to the excess of free carbon dioxide. In this state the iron compound is not filterable, because it remains in solution as a carbonate, even at pH above 7. Birm, acting as a catalyst between the oxygen and the soluble iron compounds, enhances this reaction and produces ferric hydroxide which precipitates and may be easily filtered. The physical characteristics of the Birm provides an excellent filter media which is easily cleaned by periodic backwashing. After backwashing to remove the precipitant from the bed of Birm, the filter is again ready for service flow. Birm is unique in that it is not consumed in the iron removal operation and, therefore, offers a tremendous economic advantage over many other iron removal methods under the proper conditions.

Other advantages of using Birm include long material life with relatively low loss due to attrition. Birm also performs in a wide temperature range and removal efficiency is extremely high. In many cases there are no chemicals required when utilizing Birm because Birm does not need regeneration. Simple backwashing readies the bed for service flow. Negligible labour costs are involved because only periodic backwashing is required. When considering Birm for an iron removal application, it is necessary that the water to be treated contains no oil or Hydrogen Sulphide. Organic matter should not exceed 4 - 5 ppm with a pH of 6.8 or more. If the influent water has a pH of less then 6.5, dissolved chemicals such as soda ash may be used to raise the pH. A water having a low D.O. level may be pre-treated by aeration to gain proper Dissolved Oxygen levels.

Birm is furnished in two grades, Regular and Fine. Regular Birm is generally recommended for industrial and municipal installations. Fine Birm is recommended for domestic application, or in cases where the available backwash water is limited. Bed expansion of 35 - 50% in backwash water is required for best results. 10 - 12 gpm per square foot are required for backwashing Regular Birm. 8 - 10 gpm per square foot are required for Fine Birm.

Birm may also be used for manganese removal with the same degree of dependability as when used for iron removal. In these applications the water to be treated must have a pH of 8.0 - 9.0 with a D.O. content of 15% of the total manganese and iron expressed in ppm. All other conditions remain the same for either manganese of iron removal.

Advantages

- Under the proper conditions, no chemicals to purchase for maintenance. Regeneration not required.
- Iron removal efficiency is extremely high.
- Negligible labour cost - only periodic backwashing required
- Durable material with a long life and wide temperature range.
- Weighs only 48 - 50 lbs/cu. ft.

Physical Properties

Regular Birm	Fine Birm
Color: Black	Color: Black
Density: 47 - 50 lbs/cuft	Density: 47 - 50 lbs/cuft
Effective Size: 0.81 m.m.	Effective Size: 0.48 m.m.
Uniformity Coefficient: 1.72	Uniformity Coefficient: 2.14

Conditions For Operation

pH: 6.64
Bed Depth: 30 - 36 inches
Backwash Rate: 10 gpm per square foot
Bed Expansion: Freeboard 30 - 50% of bed depth
Service Flowrate: 5 gpm per square foot, possible higher depending on local conditions

MANGANESE GREENSAND

It would be in place to compare with a short technical description of Manganese Greensand, as this will bring about some interesting relationships in the treatment field which in turn will confirm the basis for our technology on non chemical water treatment.

Manganese Greensand is formulated from a glauconite greensand which is capable of removing iron, manganese, and hydrogen sulphide from water through oxidation and filtration. Soluble iron and manganese are oxidized and precipitated by contact with higher oxides of manganese on the greensand granules. The hydrogen sulphide is eliminated by oxidation to sulphate and an insoluble precipitate. Precipitates are then filtered and removed by back washing. When the oxidizing capacity power of the Manganese Greensand bed is exhausted, the bed has to be regenerated with a weak Potassium Permanganate (KMnO4) solution this restoring the oxidizing capacity of the bed. Two to four ounces of Potassium Permanganate, in solution, per cu.ft. of Manganese Greensand is considered sufficient for normal regeneration. It is advisable to vigorously backwash and regenerate the bed before its oxidation capacity is totally exhausted.

Capacity/cu.ft.:	Iron alone...700 grains (12,000 gal of water containing 1 ppm iron/cu.ft)
	Iron and manganese...400 grains (7,000 gal of water containing 1 ppm iron and manganese/cu.ft.)
	Hydrogen sulphide...230 grains (4,000 gal of water containing 1 ppm hydrogen sulphide/cu.ft.)

Advantages

- Iron removal over a wide pH range
- Effective removal of hydrogen sulphide in addition to iron and/or manganese
- No harmful effects from a chlorine feed
- Low attrition loss for long bed life

Physical Properties

Color: black
Density: 85 lbs/cu.ft.
Effective size: 0.30 - 0.35 m.m.
Uniformity coefficient: 1.5
Mesh Size: 16 - 60
Attrition Loss Per Year: 2%

Conditions for Operation

pH: 6.0 - 8.8 Bed Depth: 30 inches Freeboard: 35 - 50% of bed depth
Backwash Flow Rate: 8 - 12 gph/sqft
Service Flow Rate: 5 gpm/sqft - intermittent flow possible
Maximum Temperature Raw Water: 100 degrees F to 37.5 degrees C
Maximum Practical Limit of Iron (FE++) or Manganese (MN++) in raw water: 15 ppm
Maximum Practical Limit of Hydrogen Sulphide (H_2S): 5 ppm
Minimum of 4 ppm Alkalinity for each ppm of iron or manganese.

ORGANICALLY BOUND IRON

We should be so lucky that all water is the same. Everything is not cast in stone and not all water sources are the same. Some of the observations we have to do when treating water may make us turn away from one type of treatment onto another. One example worth mentioning is dissolved iron in water with organic contaminants or what we call "organically bound iron". Iron in itself through our technology is easy to oxidize and treat out, in fact its a "piece of cake". However, when bound with organics the iron molecule is enclosed in a shell of organic matter which is very difficult to break away. Chlorine does an excellent job of this as does potassium permanganate. Yet a new method is the use of ozone. The mixer valve is again the main component in such a system as it provides a high quality mix and extremely quick reaction time.

The biggest asset is that the dosage of chemicals can be drastically reduces in most cases to only less than 10% of what was previously used, but what is most interesting is our results with the use of ozone. Some systems are on test at the present time and we are in the process of forming new sizing criteria and new levels of ozone dosage simply because of the advanced method of contacting with the "pressure reactor".

Same results are apperent when mixing a flocculating agent. The formation of the "floc" can be accelerated to a point where an existing, undersized treatment facility suddenly is back in business with only minor and inexpensive changes.

CONCLUSION

We have not yet converted "the world" to completely chemical free in the water treatment area and it may not ever be possible. Many new toxic chemical contaminates are being introduced into our raw water sources on an ongoing basis. But out in this great west we are still blessed with a relatively good supply of water and we should do our best to keep it this way. Eliminate or reduce dramatically the use of chemicals. Keep our standards high on potable water. Use water wisely. Take what mother nature gives us, use it and enjoy it, be careful with it and return it after use in a reasonable conditions. Originally there was no charge for the water, but we have turned water into such a commodity and we have incurred great cost already to make use of water because water is a way of life. And think of it this way "Water is Life".

DESIGN OF SHALLOW INSULATED UTILITY LINES
"A REVIEW"

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Senior Research Engineer - Construction Materials
Dow Chemical Canada Inc.

INTRODUCTION

This paper review Canadian and Scandinavian experience with using rigid board thermal insulation (specifically closed cell, extruded, expanded polystyrene insulation) to prevent the frost attack and freezing of shallow, buried underground utility lines. The shallow placement and insulation of these services in frost susceptible, non frost susceptible and rocky soils in seasonal climatic areas is investigated. No discussion is presented on other, equally valid systems or means of protecting such services from frost attack. Neither is the delivery of utility services in permafrost regions discussed.

The intent of this paper is to serve as a design guideline for the sanitary engineer by providing design charts and worked examples. It makes no attempt to discuss or substantiate the theory behind the development of the design charts or to compare the different design guidelines presented, but rather, serves as a compilation and review of design information previously documented elsewhere [1,2,3]. Such information, often published in a foreign language, has not until now, been readily usable by Canadian engineers.

SOIL INSULATION

Before we can begin to discuss the use of soil insulation techniques to prevent buried utility lines from freezing, we must understand what soil insulation is, how it works, and what properties it must have to perform reliably year after year in harsh soil environments.

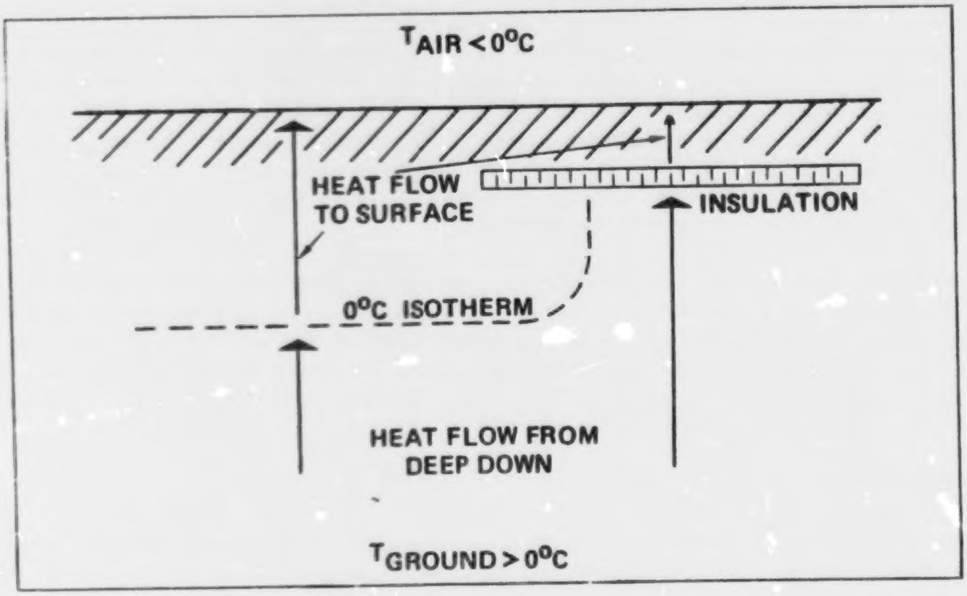
WHAT IS SOIL INSULATION?

Simply stated, soil insulation can be defined as the use of a thermal insulation material to prevent the soil beneath from FREEZING in seasonal climatic regions and THAWING in northern permafrost climatic regions.

HOW DOES SOIL INSULATION WORK?

In seasonal areas, soil insulation works basically in the same manner as insulation in the wall of your house, viz., it keeps the cold out and the warmth in! By placing a layer of thermal insulation material in the ground, one can control the effect of freezing air temperatures and hence control frost penetration into the soil beneath the insulation layer. Conversely in permafrost regions, the layer of thermal insulation controls thawing of frozen soils beneath by warm summer air temperatures and/or any surface activities which may have damaged the overlying protective active soil layer. The key word in the two preceding sentences is 'CONTROL'. In seasonal regions, if the insulation layer is thick enough (read high thermal resistance) and wide enough, freezing can be completely eliminated from the soil beneath the insulation layer. This is accomplished by a horizontal insulation layer restricting heat flow from the soil beneath to the ground surface above, Figure 1. It is well known that soil temperatures deep below the ground surface remain relatively constant year round, above freezing [4,5]. A layer of insulation placed in the ground isolates this soil from the effects of climate and thus allows for a warm region of soil temperatures to exist beneath it (known as a heat bulb). As one decreases the thickness and width of the insulation layer, obviously the soil temperatures beneath it will also decrease. The task then is to choose the correct thickness and width of insulation for the location and climate being considered.

FIGURE 1
Effect of Insulation on Frost Depth

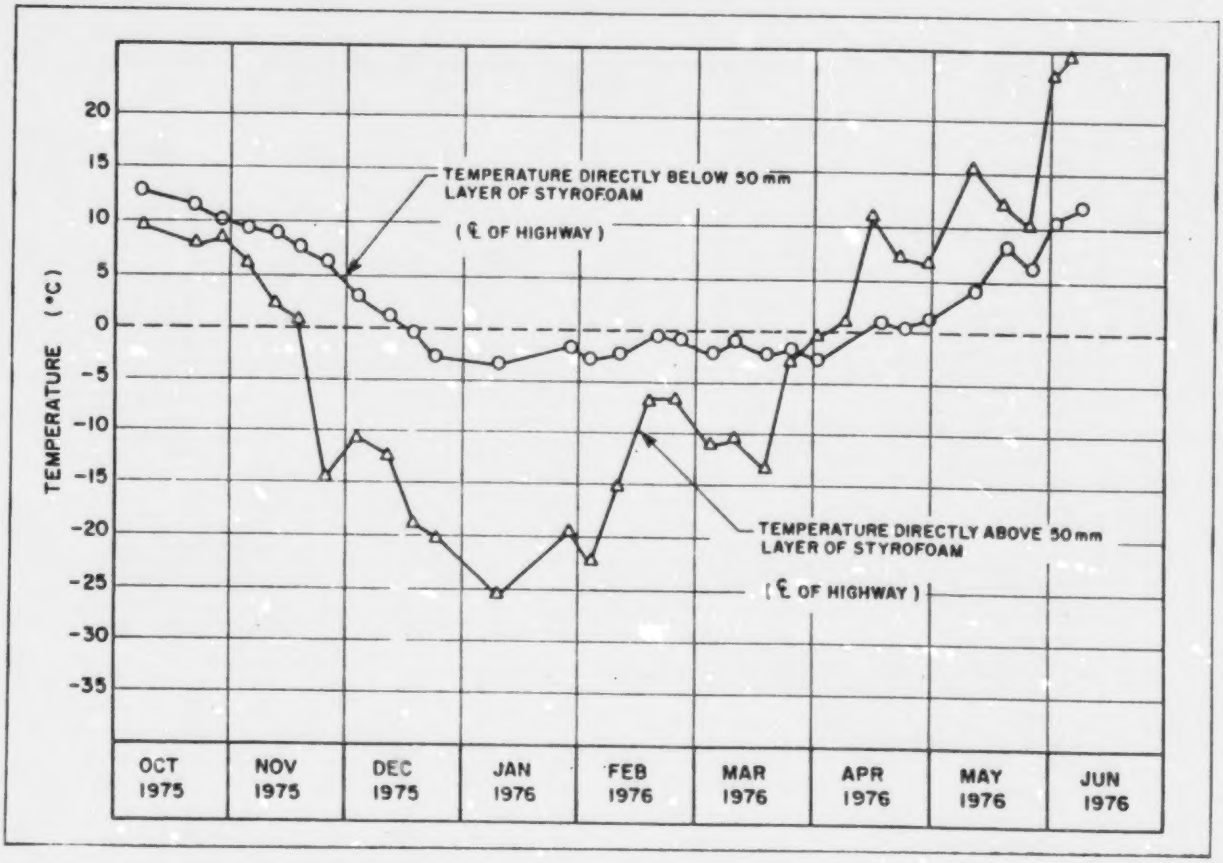


By preventing freezing of soil beneath a layer of insulation, the damaging effects of frost heave can be eliminated. Hence soil insulation is often used under structures such as highways and railways to prevent heaving of the surface, Figure 2. Likewise insulation can be placed over shallowly placed utilities to prevent the freeze-up of their contents.

WHAT PROPERTIES DOES SOIL INSULATION REQUIRE?

Since the soil insulation acts as the prime defence against a possible freeze-up of a utility line, great care is needed in selecting it. Heavy loads, soil or structures, can compress a soil insulation material, rendering it useless. Water adsorption will reduce its thermal resistance or insulating value. Freeze-thaw cycling can tear an insulation apart. When used in a soil environment, an insulating material must resist such destructive forces. Designers must select the best suited insulation available. The requirements are high compressive strength, resistance to water absorption, resistance to freeze-thaw action and a high, long lasting thermal resistance. When selecting an insulating material, one can compare their properties and performance under laboratory test conditions or investigate past field performance data on the insulation product. Ideally, one does both.

FIGURE 2
Typical Temperatures Taken Directly Above and Below a 50 mm
Layer of STYROFOAM Insulation at the Centerline (6)



PROFORMANCE OF EXTRUDED, EXPANDED POLYSTYRENE AS SOIL INSULATION

Properties of STYROFOAM* and ROOFMATE* brand insulations, two of the best known and researched extruded, expanded polystyrene insulations available today, appear in Table 1. To withstand high compressive forces without damage or loss of thermal resistance, there are various strength grades available ranging from 210 - 690 kPa (30 - 100 psi) vertical compressive strength. They have a high thermal resistance, RSI = 0.87 per 25 mm (R = 5.0 per inch). A dense, tough surface skin and a closed cell composition make both highly resistant to moisture absorption. Figure 3 and Table 2 compare their water resistance performance with that of other insulation materials in laboratory and field environments respectively. They will not sustain mold growth or decay, have no food value for plant or animal life, and have established a proven record of success in preventing sub-soil frost action in utility, highway, railroad, building foundation and other applications (6,7).

* Trademark of the Dow Chemical Co.

TABLE 1

Properties of STYROFOAM® Brand Insulation

	STYROFOAM* SM		STYROFOAM* HI-100		STYROFOAM* HI-40		STYROFOAM* HI-60		ROOFMATE*	
Thermal Resistance ASTM C-518-76 C-177-76	Ft^{2}Hr°F/Btu—in 5.0 M^2°C/W-25 mm 0.87		5.0 0.87		5.0 0.87		5.0 0.87		5.0 0.87	
Maximum Operating Temperature	F 165	C 74	F 165	C 74	F 165	C 74	F 165	C 74	F 165	C 74
Linear Thermal Coefficient of Expansion ASTM D696-79	in/in/F° 3.5 x 10^{-5} mm/m/C° 6.3 x 10^{-2}		in/in/F° 3.5 x 10^{-5} mm/m/C° 6.3 x 10^{-2}		in/in/F° 3.5 x 10^{-5} mm/m/C° 6.3 x 10^{-2}		in/in/F° 3.5 x 10^{-5} mm/m/C° 6.3 x 10^{-2}		in/in/F° 3.5 x 10^{-5} mm/m/C° 6.3 x 10^{-2}	
Water Absorption (% by volume) max. ASTM D 2842-69 (1975)	0.7%		0.7%		0.7%		0.7%		0.7%	
Water Vapour Permeance (Maximum)③ ASTM E96-80	perm 0.75	ng Pa^{-1} sec^{-1} m^{-2} 43	perm 0.6	ng Pa^{-1} sec^{-1} m^{-2} 35	perm 0.6	ng Pa^{-1} sec^{-1} m^{-2} 35	perm 0.6	ng Pa^{-1} sec^{-1} m^{-2} 35	perm 0.6	ng Pa^{-1} sec^{-1} m^{-2} 35
Capillarity	None		None		None		None		None	
Compressive Strength (min.) ASTM D-1621-73 (1979)	psi① 30	kPa① 210	psi① 100	kPa① 690	psi① 40	kPa① 275	psi① 60	kPa① 415	psi① 35	kPa① 240
Tensile Strength (Typical) ASTM D-1623-78 (Method A)	psi 50	kPa 350	psi 125	kPa 860	psi 70	kPa 480	psi 85	kPa 590	psi 60	kPa 415
Shear Strength (Typical) ASTM C-273-61	psi 35	kPa 240	psi 50	kPa 350	psi 40	kPa 275	psi 45	kPa 310	psi 35	kPa 240
Flexural Strength (Typical) ASTM C-203-85	psi 50	kPa 350	psi 85	kPa 585	psi 70	kPa 480	psi 85	kPa 585	psi 60	kPa 415

① At 10% deformation or yield.

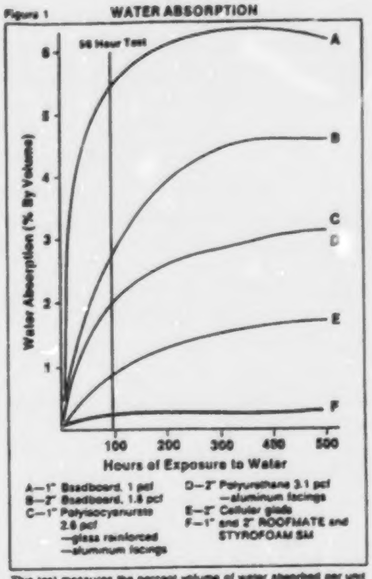
② At 5% deformation or yield.

③ For 25 mm or 1 inch thickness.

FIGURE 5

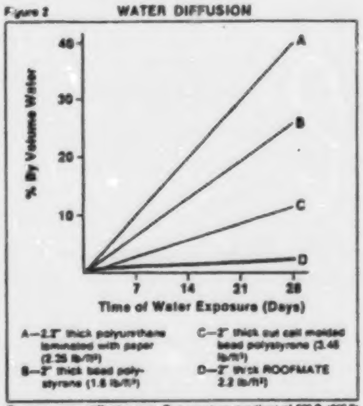
Water Adsorption Characteristics of Various Insulation Materials Under Laboratory Conditions

Figure 1 **WATER ABSORPTION**



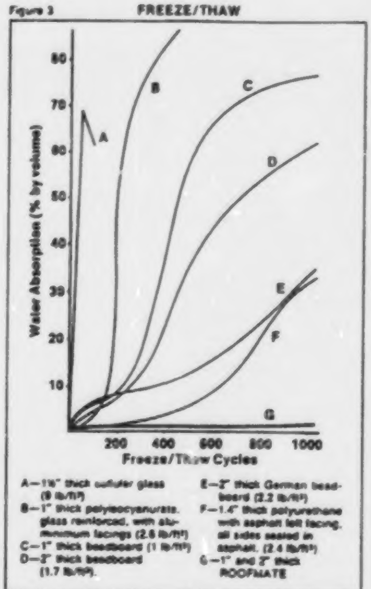
This test measures the percent volume of water absorbed per unit volume of foam. The 96 hour test is described in ASTM D 2842-69. Since longer periods of submersion can occur in some applications the test was run for 500 hours (21 days).

Figure 2 **WATER DIFFUSION**



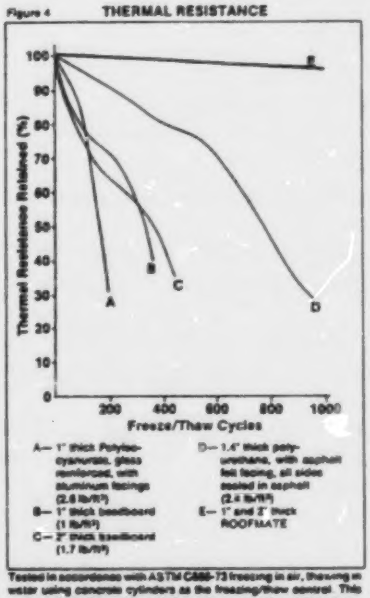
German water diffusion test. Temperature gradient of 50° C (90° F) was maintained on each sample throughout the 28 day test. All samples were 5-6 cm. ≅ 2 inches. thick.

Figure 3 **FREEZE/THAW**



Tested in accordance with ASTM C 666-73 freezing in air, thawing in water using concrete cylinders as the freeze/thaw control. This test is useful in determining relative performance of insulations in applications where freezing and thawing occur in the presence of water

Figure 4 **THERMAL RESISTANCE**



Tested in accordance with ASTM C666-73 freezing in air, thawing in water using concrete cylinders as the freezing/thaw control. This test is useful in determining relative performance of insulations in applications where freezing and thawing occur in the presence of water.

TABLE 2

Field Performance of Various Insulations Used In Below-Grade Applications (12)

					Properties of Insulations After Years in Service		
Insulation	Application	Thickness cm (in.)	Density, kg/m³ (lb/ft³)	Time in Service, years	Water Absorption % by volume	W/m·K K, (Btu·in./h·ft²·deg F)	Published λ, W/m·K K, (Btu·in./h·ft²·deg F)
A. EXTRUDED POLYSTYRENE SKINBOARD							
1. Midland, Mich.	highway insulation	2.54 (1)	40.04 (2.50)	10	4.24%	0.0311 (.220)	0.0288 (0.20)
2. Midland, Mich.	highway insulation	2.54 (1)	40.04 (2.50)	1.7	0.26%	0.0292 (.202)	0.0288 (0.20)
3. Ontario, Canada	soil insulation	5.08 (2)	38.28 (2.39)	5	0.36%	0.0303 (.210)	0.0288 (0.20)
4. Ontario, Canada	soil insulation	3.81 (1½)	42.45 (2.65)	5	0.44%	0.0318 (.221)	0.0288 (0.20)
5. Midland, Mich.	below-grade perimeter insulation	3.81 (1½)	33.00 (2.06)	2	0.56%	0.0297 (.206)	0.0288 (0.20)
6. Midland, Mich.	below-grade perimeter insulation	5.08 (2)	33.64 (2.10)	4	1.10%	0.0307 (.213)	0.0288 (0.20)
7. Midland, Mich.	below-grade perimeter insulation	3.81 (1½)	35.40 (2.21)	6	1.20%	0.0333 (.231)	0.0288 (0.20)
8. Midland, Mich.	below-grade perimeter insulation	3.81 (1½)	39.72 (2.48)	7	1.20%	0.0318 (.221)	0.0288 (0.20)
9. Midland, Mich.	below-grade perimeter insulation	3.81 (1½)	33.48 (2.09)	8	0.70%	0.0317 (.220)	0.0288 (0.20)
B. MOLDED BEAD POLYSTYRENE							
1. Midland, Mich.	highway insulation	2.54 (1)	20.02 (1.25)	1.7	0.41%	0.0371 (.257)	0.0346 (0.24)
2. Midland, Mich.	highway insulation	2.54 (1)	20.34 (1.27)	10	9.55%	0.0507 (.351)	0.0346 (0.24)
C. POLYURETHANE (without skins)							
1. Midland, Mich.	highway insulation	2.54 (1)	30.43 (1.90)	1.7	4.2%	0.0343 (0.238)	(0.13–0.16) 0.0187 to 0.0230
2. Midland, Mich.	highway insulation	2.54 (1)	30.27 (1.89)	10	20.31%	0.0484 (0.336)	(0.13–0.16) 0.0187 to 0.0230
3. Midland, Mich.	highway insulation	2.54 (1)	30.43 (1.90)	15	17.08%	0.0629 (0.436)	(0.13–0.16) 0.0187 to 0.0230

ADVANTAGES OF INSULATED, SHALLOW BURIED UTILITY LINES

Briefly stated the use of insulation as thermal protection against freeze-up of shallow buried utility lines has numerous advantages. They can be summarized as follows:

1. Less excavation is required with utility lines placed above the level of frost penetration and depending on the soil type to be excavated or blasted, cost savings can be substantial.

2. Because of the shallow depth at which the utility may be placed, cost reductions in maintenance/service facilities (ie: service manhole structure heights, etc.) are possible.

3. Deep excavation shoring costs may be eliminated.

4. Costs for select backfilling materials may be reduced as less are required with insulated shallow utility services.

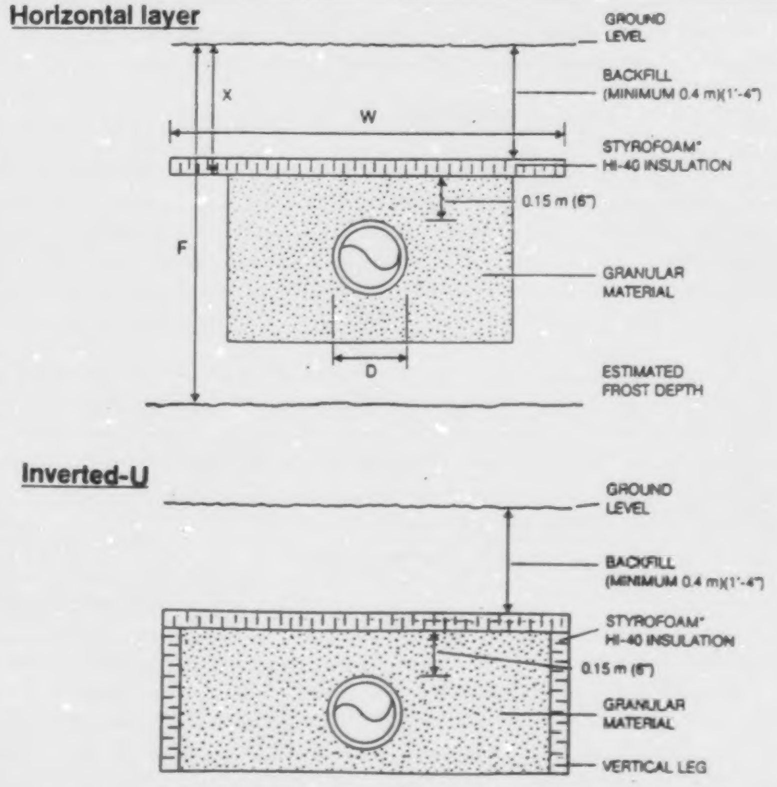
5. By using the shallow insulation technique, it is no longer necessary to lower an existing line when the removal of part of the protective soil cover occurs, thereby saving construction dollars.

CANADIAN DESIGN CASE ONE

The empirical design method presently in use by the Dow Chemical Company is illustrated in Figure 4. Derived from two-dimensional finite element computer model for heat loss predictions, results from this method, though slightly conservative, have proven satisfactory in field applications dating as far back as 1965 [8]. Its use is restricted to seasonal climatic areas in sand, silt and clay soils.

FIGURE 4

Dow Chemical Empirical Design Methods for Insulation Requirements to Protect Shallow Buried Utilities in Soil


Horizontal layer
GROUND LEVEL
X
W
BACKFILL (MINIMUM 0.4 m)(1'-4")
STYROFOAM* HI-40 INSULATION
0.15 m (6")
F
GRANULAR MATERIAL
D
ESTIMATED FROST DEPTH
Inverted-U
GROUND LEVEL
BACKFILL (MINIMUM 0.4 m)(1'-4")
STYROFOAM* HI-40 INSULATION
0.15 m (6")
GRANULAR MATERIAL
VERTICAL LEG

EQUATION 1: WIDTH OF INSULATION: $W=3D+2[F-(X+D+0.5)]$ For Imperial calculation
$W=D+2(F-X)-0.3$ For Metric calculation

Where W = width of insulation (meters) (ft)
D = pipe diameter (meters) (ft)
X = insulation depth (meters) (ft)
F = estimated frost depth (meters) (ft)

Two different insulation configurations are possible. The first involves the placement of a horizontal layer of insulation just above the utility. This configuration is perhaps better suited for utilities built in mild climates or where they are placed only slightly above the depth of frost penetration. The width of soil insulation cover needed to prevent freeze-up of the utility is given by Equation 1 shown in Figure 4. All necessary heat required to prevent the utility from freezing is gained from geothermal contributions. The only information required by the designer is the approximate depth of placement of the insulation, the diameter of the utility and the expected frost penetration in the area. The width of insulation calculated prevents excess heat loss from the sides and hence freezing from the sides.

The thickness of insulation employed by this method prevents heat loss vertically and hence freezing of the utility from above. Table 3 gives the thickness of STYROFOAM brand insulation required based on the expected depth of backfill cover over the insulation and the design air-freezing index (9) for the location of the project.

TABLE 3

Thickness of STYROFOAM® Brand Insulation Required to Prevent Freezing of Shallow Buried Utilities

Amount of Backfill over the Insulation (feet)	Design Freezing Index (°F-Days)						
	1500	2000	2500	3000	3500	4000	4500
2'-0"	2.0"	2.5"	3.0"	3.5"	4.0"	4.5"	5.0"
3'-0"	1.5"	2.0"	2.5"	3.0"	3.5"	4.0"	4.5"
4'-0"	1.0"	1.5"	2.0"	2.5"	3.0"	3.5"	4.0"
5'-0"	1.0"	1.0"	1.5"	2.0"	2.5"	3.0"	3.5"
6'-0"	1.0"	1.0"	1.0"	1.5"	2.0"	2.5"	3.0"
7'-0"			1.0"	1.0"	1.5"	2.0"	2.5"
8'-0"				1.0"	1.0"	1.5"	2.0"
9'-0"					1.0"	1.0"	1.5"
10'-0"						1.0"	1.0"

Thickness of Insulation in Inches

Amount of Backfill over the Insulation (m)	Design Freezing Index (°C-Days)						
	850	1125	1400	1675	1950	2225	2500
0.6	50.0	65.0	75.0	90.0	100.0	115.0	125.0
0.9	40.0	50.0	65.0	75.0	90.0	100.0	115.0
1.2	25.0	40.0	50.0	65.0	75.0	90.0	100.0
1.5	25.0	25.0	40.0	50.0	65.0	75.0	90.0
1.8	25.0	25.0	25.0	40.0	50.0	65.0	75.0
2.1			25.0	25.0	40.0	50.0	65.0
2.4				25.0	25.0	40.0	50.0
2.7					25.0	25.0	40.0
3.0						25.0	25.0

Thickness of Insulation in mm

For designs in more severe climates or with very little overlying ground cover, the width of horizontal insulation determined by Equation 1 becomes large and as a result, restrictive. Therefore a second configuration, called the Inverted-U may be employed. When placed in this configuration, the total parimeter length of the insulation is equivalent to that which would be used if a horizontal layer was employed. Portions of the length are placed vertically downwards along the side of the utility trench. This configuration allows for the reduction in the width required and uses the vertical legs of insulation to prevent freezing of the utility from the side. The thickness of STYROFOAM brand insulation required is the same as if a horizontal configuration was used and is found in Table 3.

FROST PREVENTION METHODS IN SCANDINAVIA

When utilities are not placed below the local frost depth, thermal insulation must be employed to prevent them from freezing. Frost prevention in pipe trenches is aided by heat from the ground as well as heat given off by water or sewage lines. If the utility is placed in a frost susceptible soil, it is advantageous to cover it with a wide horizontal layer of insulation. If the soil is not frost heave susceptible, it is not necessary to prevent the ground below the pipe from freezing. However, if frost prevention is to be accomplished using ground heat alone, the soil below the pipe must be prevented from freezing.

The design aids which follow were developed by researchers on the Norwegian Committee for Frost Action in Soils and have been used extensively in Scandinavia [10]. Field research here [11] has shown that systems designed with these design aids have performed as expected. However, it has been identified [11] that there is a lack of guidelines for calculating the possible heat output from water and sewage lines installed below grade. In the case of the insulation box configuration, it has been suggested that there is insufficient information on the relationship between the thickness of the box top, sides and bottom to achieve the best possible protection and economy. Ongoing research efforts have been focused in these directions.

When using the Scandinavian design aids, one must convert the Canadian tabulation of design air freezing index from degree-days ^{0}C to the Scandinavian convention of degree-hours ^{0}C. The original design aids specified the required insulation thermal resistance in m2·K/W units. To make the charts more user friendly to Canadians, the required insulation thermal resistance has been equated in terms of extruded, expanded polystyrene insulation thickness. Note that extruded, expanded polystyrene insulation has a thermal resistance of 0.87 m2^{0}C/W - 25 mm (5.0 ft2·hr ^{0}F/btu-in).

SCANDINAVIAN DESIGN CASE ONE

The following three design aids are used when one can not rely on any heat loss from the buried utility. This case would include any pipe lines carrying very small quantities of water as well as those with intermittent or interrupted flow. Such is often the case with water lines in thinly populated areas or in resort areas where the water line is used irregularly. Figures 5 and 6 are design charts for soils of sand/gravel and clay respectively. Note that these figures can be used for insulating existing utilities which were originally placed below the depth of frost penetration but now, because of some soil cover removal, are above the expected design frost depth. Hence existing lines may be protected without complete excavation to the line to place the insulation. The designer should ensure that the line is located below the depths defined by the factor 'Z' and that the insulation is placed at least 0.5 m (1' - 8") below grade.

FIGURE 5

Horizontal Insulation Requirements to Protect Shallow Buried Utilities in Sand/Gravel

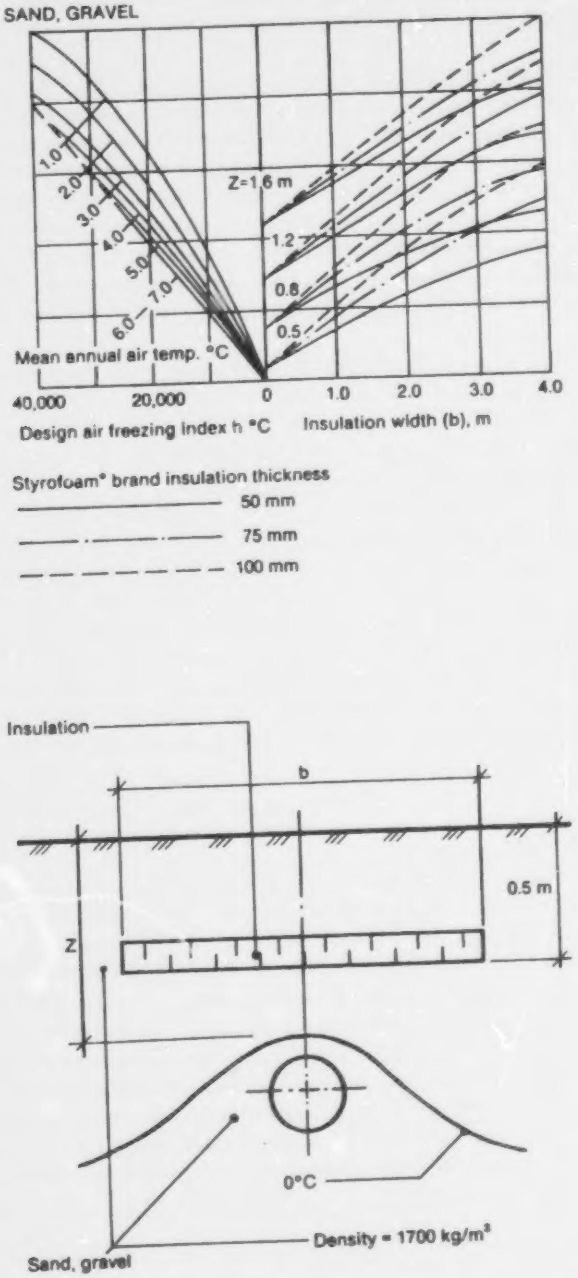
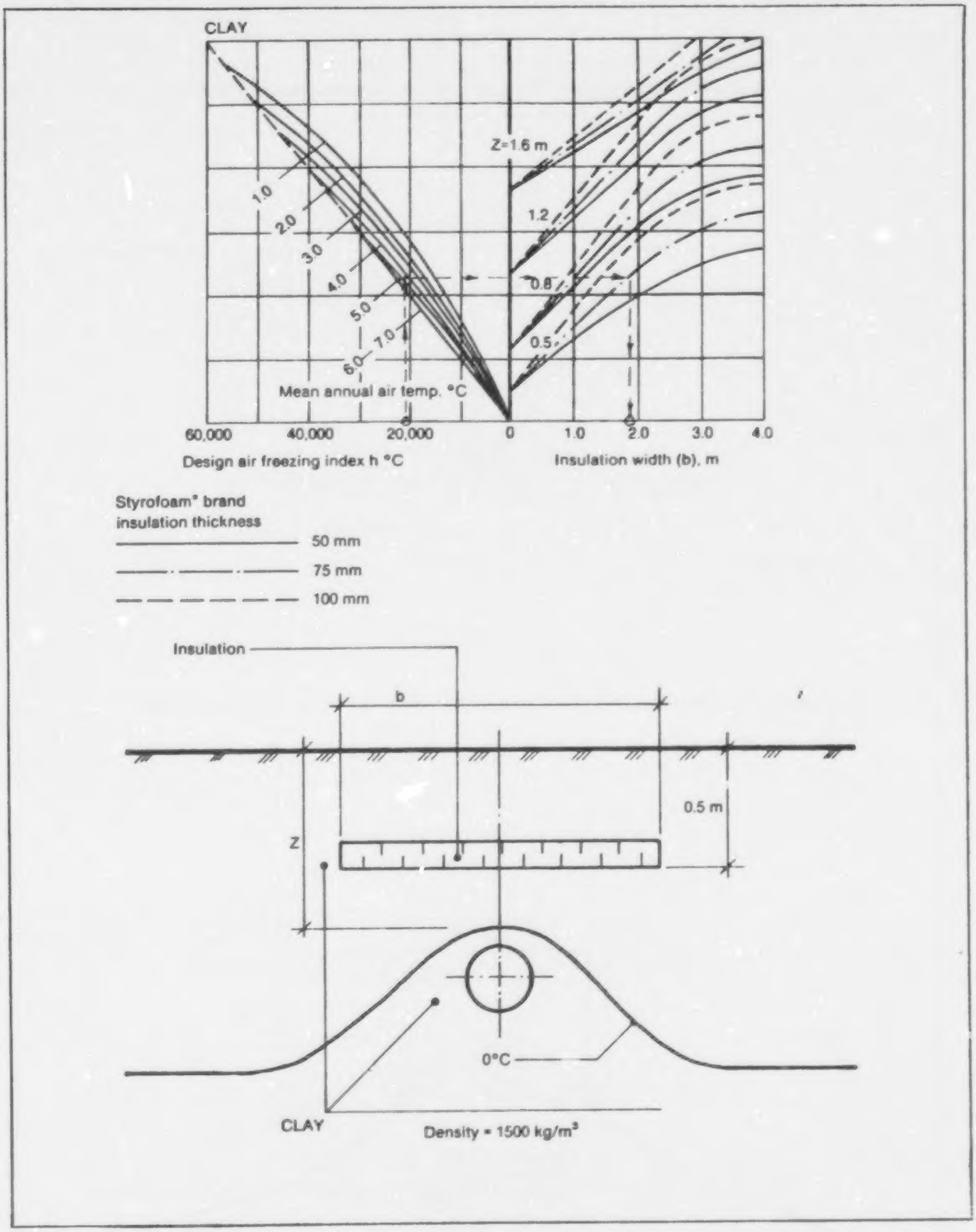


FIGURE 6

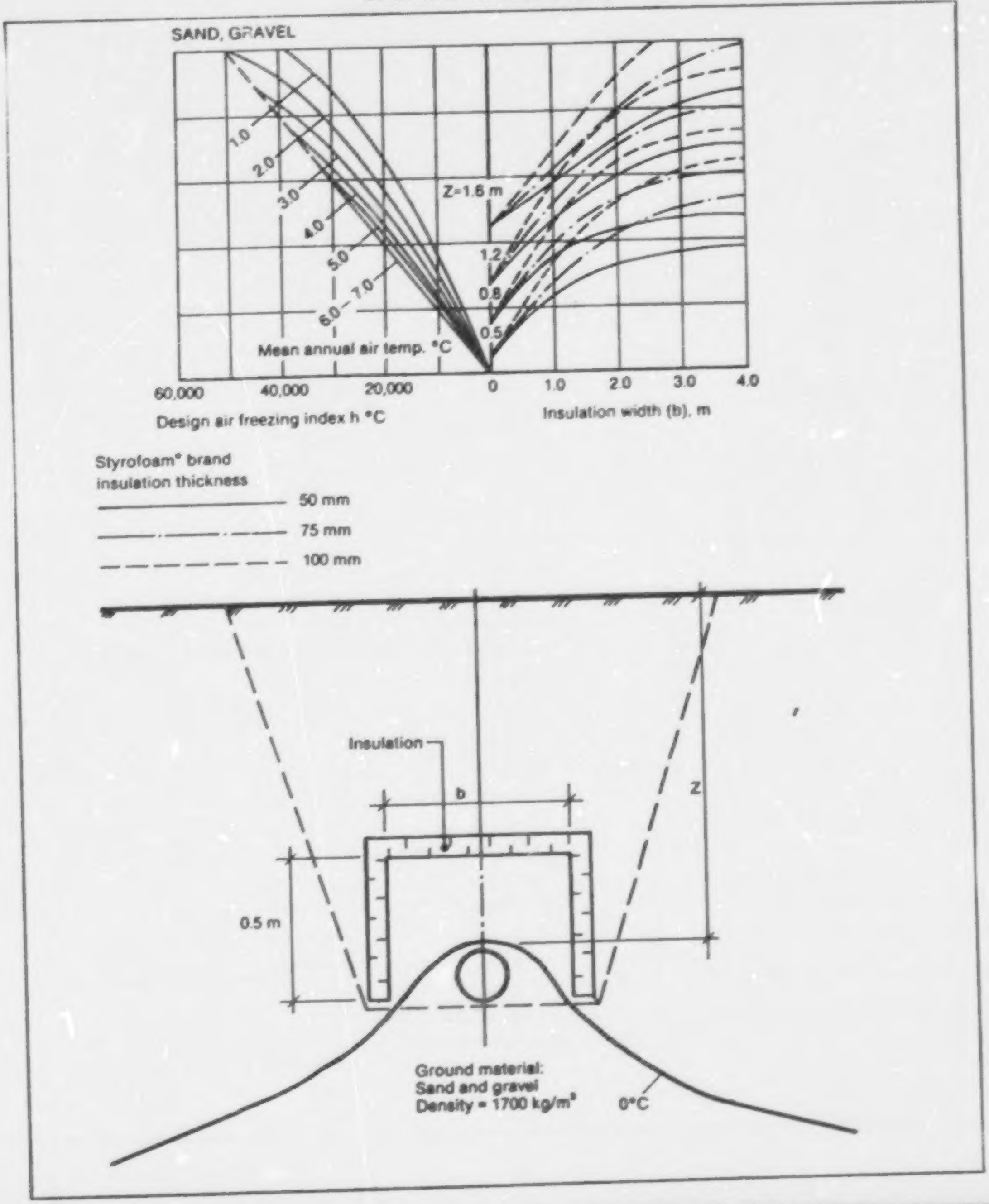
Horizontal Insulation Requirements to Protect Shallow Buried Utilities in Clay



Reductions in the required insulation width and thus reductions in excavation cost can be achieved by placing the insulation in a horseshoe configuration. Figure 7 gives the insulation width and thickness required in order to attain a frost depth of 0.5, 0.8, 1.2 or 1.6 meters [1'- 8", 2'- 8", 4'- 0" or 5'- 3"] directly below a horseshoe configuration for different freezing indices. The ground material is sand or gravel.

FIGURE 7

Horseshoe Insulation Configuration to Protect Shallow Buried Utilities in Sand/Gravel



SCANDINAVIAN DESIGN CASE TWO: INTERNAL HEAT SOURCE

In most cases, it is usually the combination of heat given off by water or sewage utilities and that provided by the ground that prevents freezing. In some cases, a heat tracing electric cable may be provided if heat generation from the utility can not be reliably utilized. Two nomograms, Figures 8 and 9, show the horizontal insulation requirements for sand/gravel and for clay soils respectively, while a third, Figure 10 shows horseshoe shaped insulation requirements for use in sand and gravel. One would use these nomograms when the insulation requirements found in Design Case One were excessive.

FIGURE 8

Horizontal Insulation and Heat Requirements to Protect Shallow Buried Utilities in Sand/Gravel

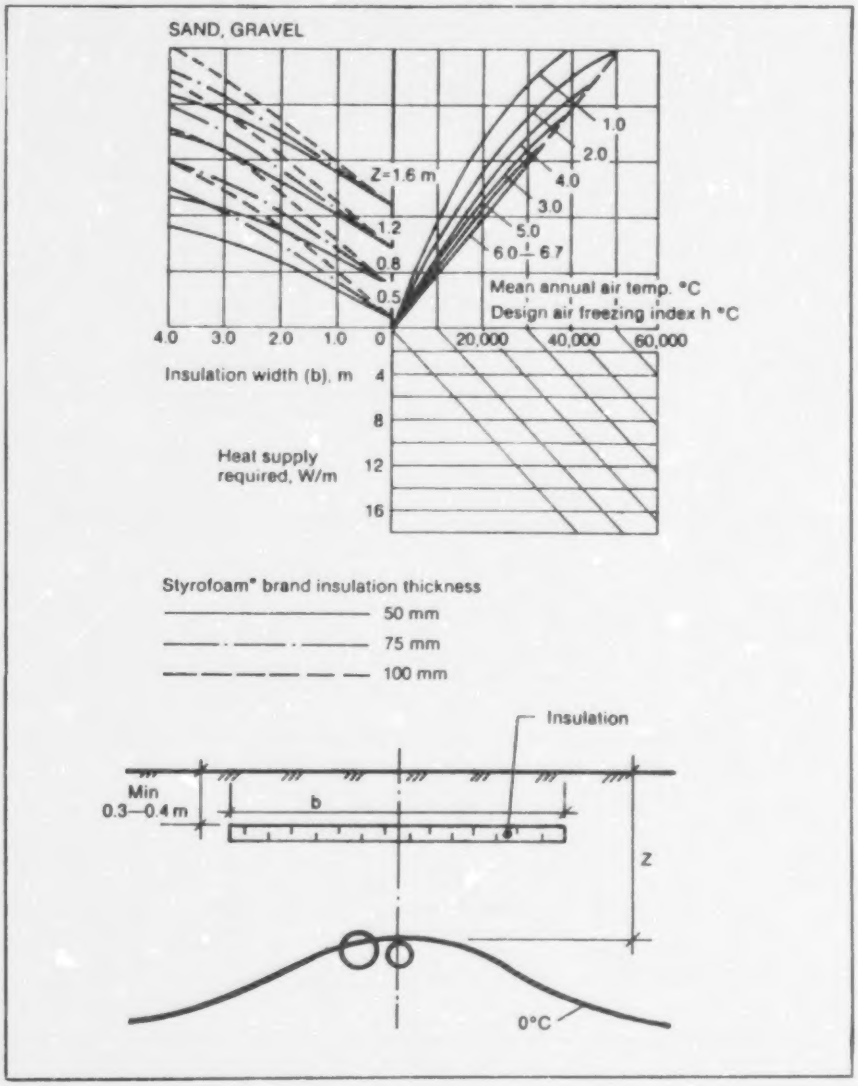


FIGURE 9

Horizontal Insulation and Heat Requirements
to Protect Shallow Buried Utilities in Clay

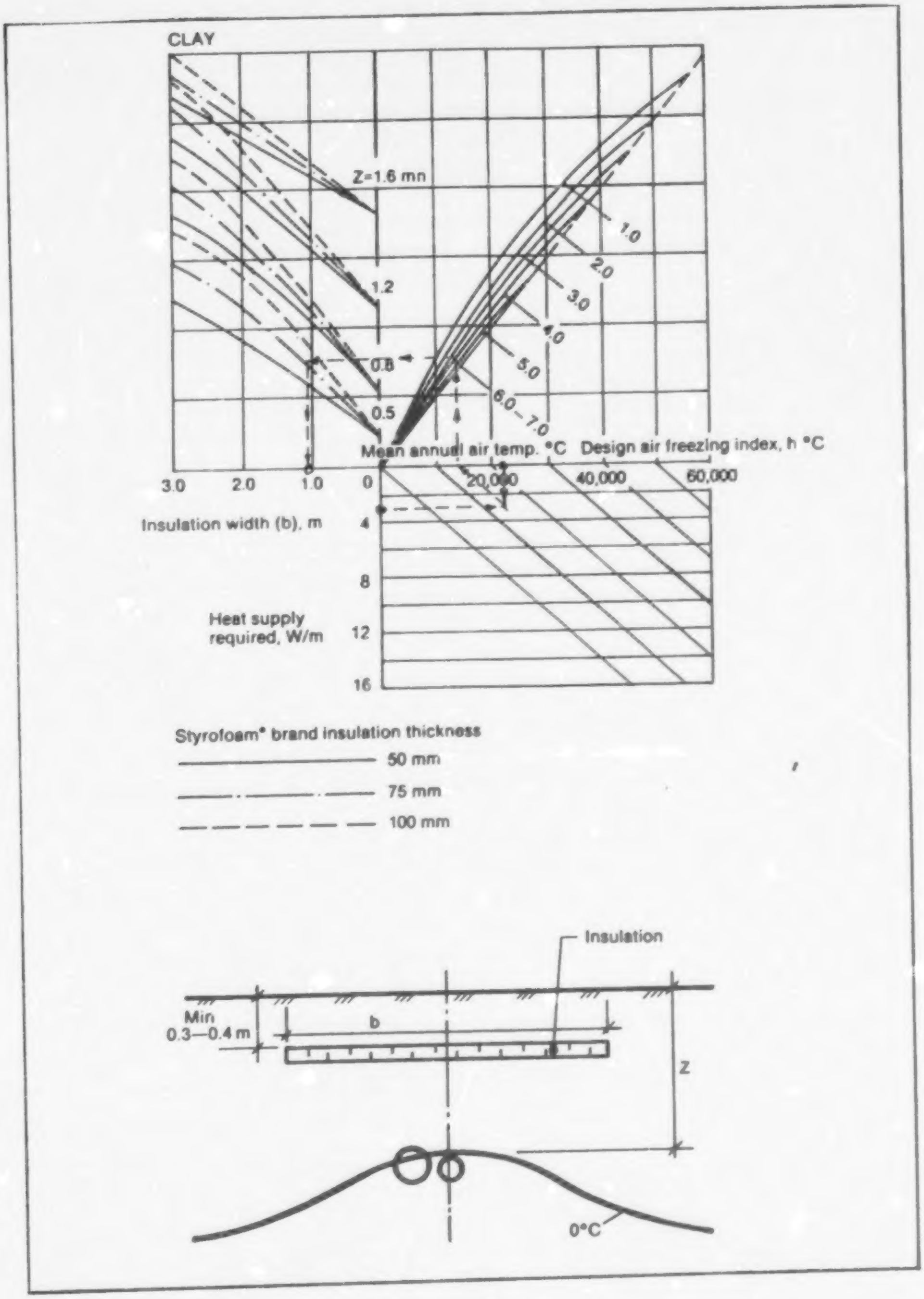
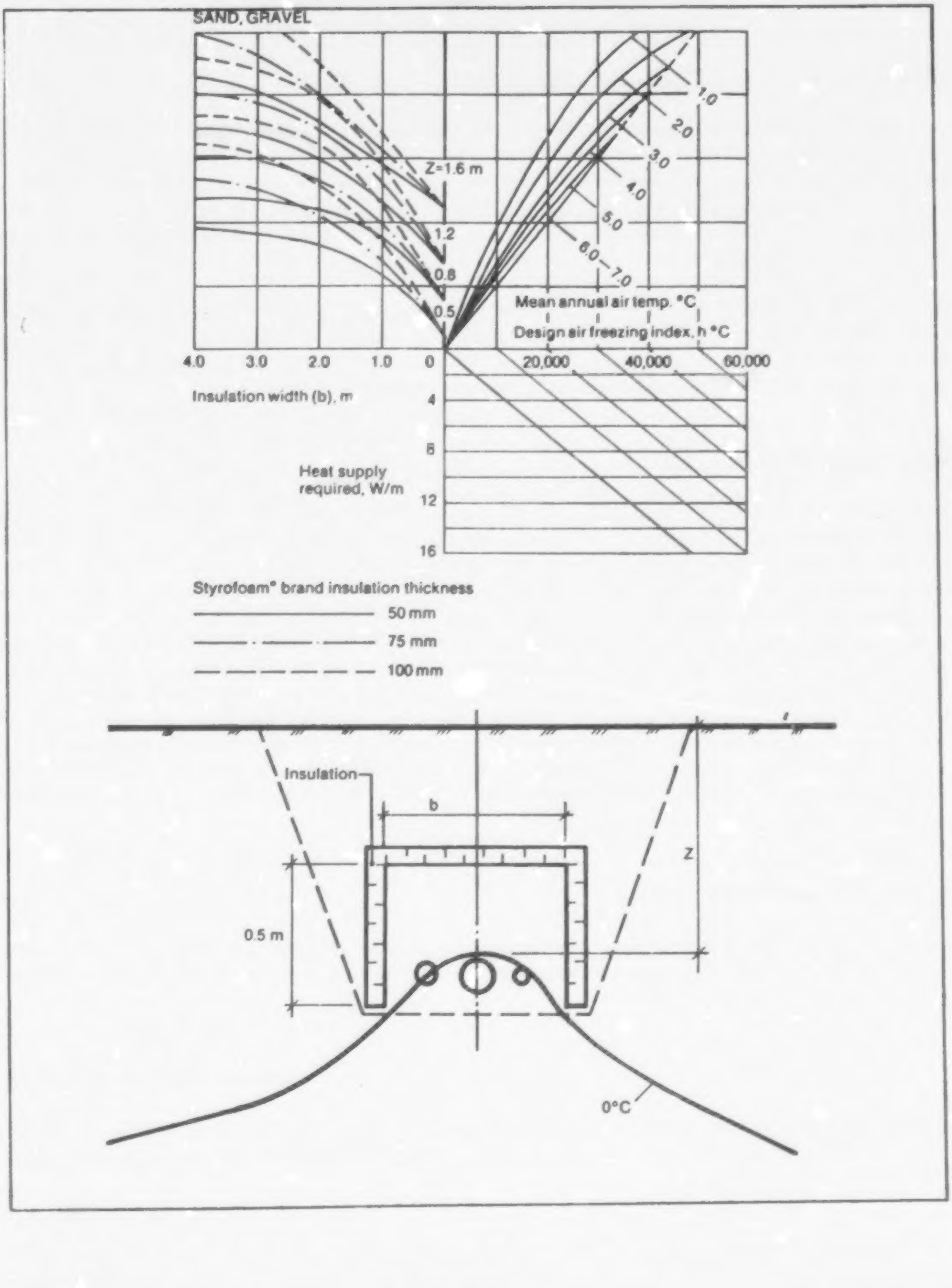


FIGURE 10

Horseshoe Insulation Configuration and Heat Requirements
to Protect Shallow Buried Utilities in Sand/Gravel



To use these design aids, one enters the charts knowing the maximum air freezing index for the geographical location of the utility and the probable required heat supply [if using an electric cable] or pipeline heat losses. Using these numbers, one then determines a dimensioning air freezing index, which, when used with the annual mean air temperature and insulation placement depth, provides the necessary insulation width and thermal resistance.

SCANDINAVIAN DESIGN CASE THREE: UTILITIES IN ROCK

In Scandinavia, more often than not, practical considerations, not the depth of frost, dictate how deep water and sewage lines are placed. The abundance of rocky terrain has lead to the development of design aids that reduce digging and blasting by utilizing the heat released by utilities by means of high quality insulation materials. Layouts in the terrain are chosen to utilize naturally occurring factors such as insulating effect of snow and placements away from roadbeds are used wherever possible.

Rock is a good thermal conductor and frost will penetrate to great depths. The use of soil insulation in rock trenches is primarily a matter of keeping the utility effluent temperatures above freezing as frost-heaves do not occur in rock. Thus freezing may occur below the utility and therefore heat losses from the utility must be kept to a minimum.

NARROW VERSUS WIDE ROCK TRENCHES

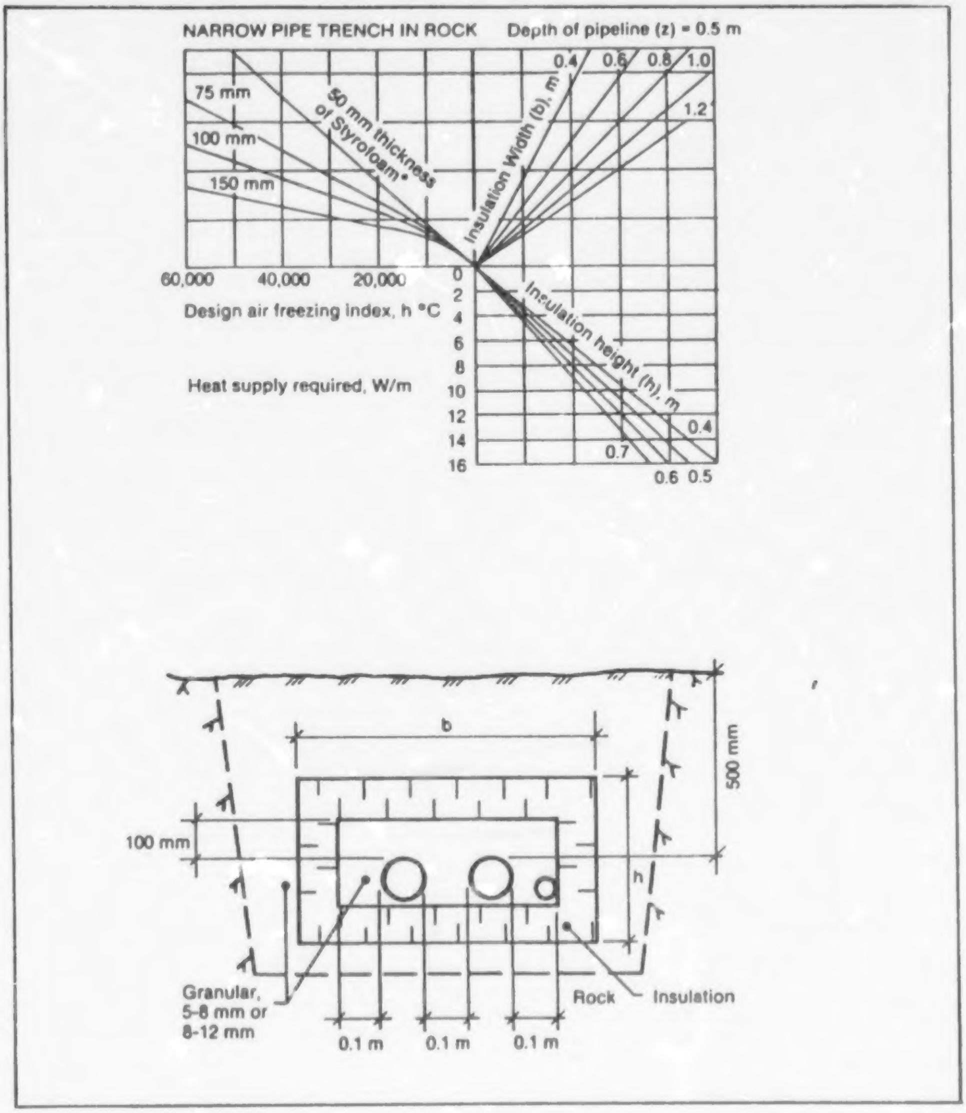
The Scandinavians distinguish between narrow and wide rock trenches. They define a narrow rock trench as one where the backfill over the utility serves mainly as structural protection for the utility and not as thermal protection. In such trenches, because of the proximity of rock in the sides of the trench and hence freezing potential from the sides, laying depth does not have a major effect on the necessary heat supply required. A wide trench is one where the backfill materials are significant for frost prevention but that the rock is still within the normal depth of frost penetration. Therefore there is a potential that the utility may freeze from below. A rock trench may be considered wide if the trench width at the surface exceeds twice the frost penetration depth of the backfill material. Note that when designing insulated utilities in rock trenches, minimizing the total width and height of the insulation box will minimizing the insulation box will minimize the required heat supply for a given thickness of insulation. The filling material between the insulation and the utility is assumed to be well compacted gravel.

UTILITIES IN NARROW ROCK TRENCHES

Figure 11 shows the required heat dissipation needed for preventing frost attack of insulated utilities in a narrow rock trench. Note that the minimum covering soil layer is 0.5 meters [1-8"].

FIGURE 11

Insulated Box and Heat Requirements to Protect Shallow Buried Utilities in Narrow Rock Trench

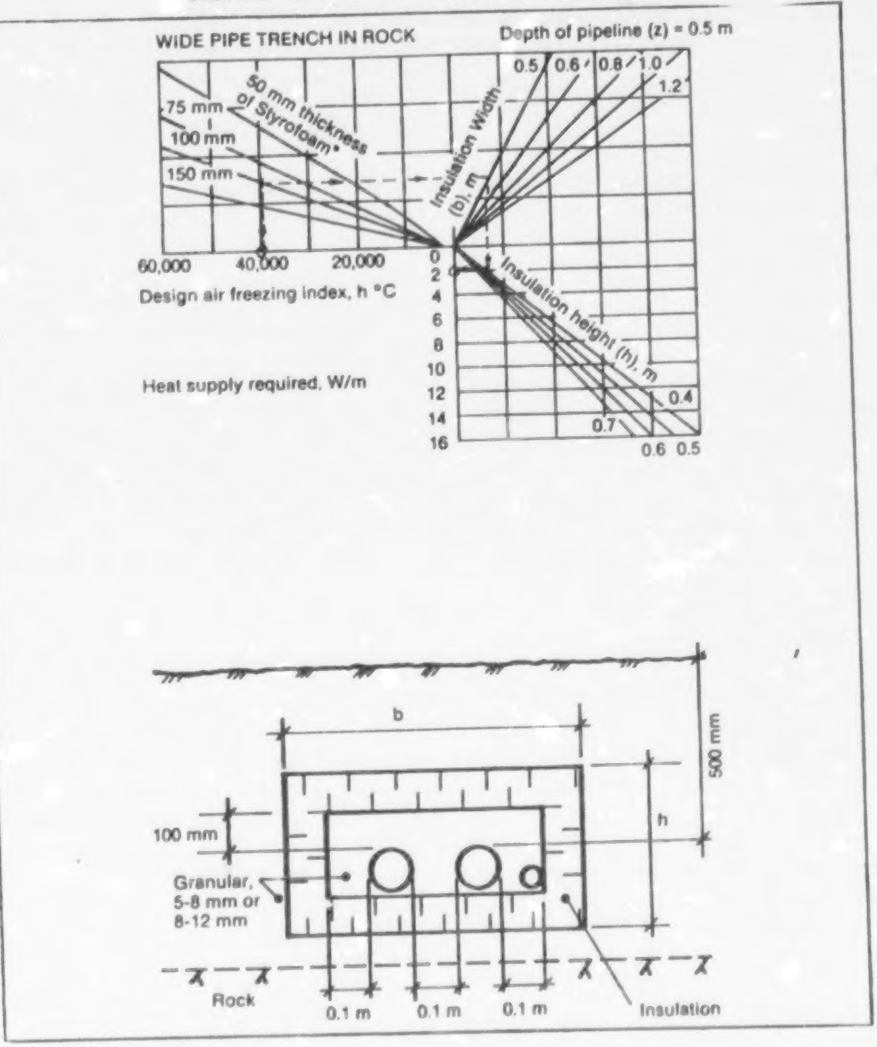


UTILITIES IN WIDE ROCK TRENCHES

Figure 12 gives the heat dissipation and insulation thickness necessary for preventing freeze-up of insulated utilities in wide rock trenches.

FIGURE 12

Insulation Box and Heat Requirements to Protect Shallow Utilities in Wide Rock Trenches



DESIGN DETAILS

Though it is next to impossible to discuss all the detailed factors in the design of the insulated shallow utility in a single paper, one of the most important is worthy of mention.

When insulation is used to reduce the laying depth of a utility pipeline, the effect of the frost bridges occurring at control devices for valves, fire hydrants, etc., must be minimized. Also, the ventilation of waste water pipelines that are insulated should be kept to an absolute minimum to avoid drafts of cold air. It is recommended that at frost bridges such as these, the insulation thickness be doubled for about one metre (3'- 6") around the inlet.

CONCLUSION

Soil insulation techniques have been well researched and documented around the world. The Canadian and Scandinavian design aids presented in this paper are only the first step in the design of a successful, cost efficient shallow insulated utility system. Designers are encouraged to investigate further the other important factors that must be considered in such a design and to examine the worked design examples appearing in the Appendix.

ACKNOWLEDGEMENT

The Scandinavian design charts presented in this paper were not developed by this author. Their development is credited to those dedicated researchers on the Norwegian Committee for Frost Action in Soils. Begun in 1970 and concluded in 1976, the Committee works have been published in 17 issues of the publication: "Frost i Jord" [Frost in Action in Soils]. Funded by the Royal Norwegian Council for Scientific and Industrial Research (NTNF) and the Public Roads Administration, the following institutions participated in the Committee research:

Division of Refrigeration Engineering
Division of Housebuilding Engineering
Division of Road and Railway Engineering
Norwegian Building Research Institute
Norwegian Road Research Laboratory
Multiconsult A/S

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APPENDIX

DESIGN EXAMPLE ONE:

A 150 mm (6") buried water line is proposed to be constructed on bedrock in the outskirts of the Manitoban town of Portage La Prairie. It is desired to place the water line under a reduced covering layer of soil without rock blasting. The design air freezing index in Portage La Prairie is 1667 degree-days °C or 3000 degree-days °F. The mean annual air temperature for Portage La Prairie is 3.0 °C or 37.5 °F. the depth of frost penetration at the proposed site is unavailable.

Your firm has been asked to submit a proposal on the design of this shallow buried, insulated water line. What initial recommendations would you provide your boss?

Knowing that the Scandinavian nomograms require the use of design freezing index in the form of degree-hours °C, we convert the Canadian value to this convention:

Design air freezing index for Portage La Prairie:

= (1667 degree-days °C) x (24 hours/day)

= 40,000 degree-hours °C

Using Figure 12, (the water line is on bedrock with minimum soil cover, therefore we assume a design using a wide rock trench) we find that if the line is placed 0.5 m (1'-8") below the final grade of the covering soil, a 100 mm (4") thickness of extruded, expanded polystyrene insulation, in a box configuration 0.5 m x 0.5 m (1'-8" x 1'-8") in size is required. Since we are only protecting one line 150 mm (6") in diameter, the dimensions of the box are adequate. Note that an electric heating cable or the minimum design heat dissipation from the line due to flow must provide at least 2 W/m of heat to ensure protection from freezing.

DESIGN EXAMPLE TWO

Near the small community of Dome Creek, British Columbia, a shallow buried water line is to be placed in frost susceptible soil (fine silts and clay). The water table is located about 1 m (3'- 4") below the surface. The design air freezing index for Dome Creek is given as 875 degree-days °C or 1575 degree-days °F. The mean annual air temperature has been documented as 4.1 °C or 39.3 °F.

You have been asked to propose the most suitable insulation configuration which would prevent freezing of the shallow buried water line and would also prevent possible frost heaves in the soil around the line. Many homes will be connected to the water main once all development has been completed; in about five years. At that time, the minimum flow rate averaged over a 24 hour period has been predicted and 3 W/m of heat generation is expected from that minimum flow.

Design air freezing index for Dome Creek using the Scandinavian dimensioning convention:

= (875 degree-days °C) x (24 hrs/day)
= 21,000 degree-hours °C

Mean annual air temperature = 4.1 °C

Minimum heat generation from line = 3 W/m when all development is completed in five years time. However, until that development occurs, no reliable source of heat generation from the line can be counted on. Therefore, the line is designed assuming no or intermittent flow only.

Using Figure 6 and assuming that the line will not be fully operational for at least 5 years, we find that if the line is buried at 0.5 m (1'- 8") below grade (keeps trench excavation above the groundwater table), a 75 mm (3") thickness of extruded, expanded polystyrene insulation is necessary to protect the line from freeze-up. The horizontal width of the insulation layer (which is placed 0.4 m or 16" below the ground surface when the pipe depth "Z" is 0.5 m or (1'- 8") is 1.8 m (6'- 0") and is wide enough to prevent the ground below it from freezing.

As the depth of insulation is only 0.4 m (16") below grade, care is required in selecting the proper strength of the insulation to resist any applied vehicle loads. If the line has been placed below a roadway, STYROFOAM* HI-60 brand extruded polystyrene insulation (415 kPa or 60 psi vertical compressive strength) is recommended.

If the design was based on utilizing the minimum 3 W/m heat dissipation from a flow in the line, one would use Figure 9. This diagram shows that the additional heat contribution of 3 W/m from the line allows for a 40 percent reduction in the width of the insulation over the line (1.1 m or 3'- 8" required instead of 1.8 m or 6'- 0").

DESIGN AND REHABILITATION CONSIDERATIONS FOR CONTROL OF SEEPAGE FROM MUNICIPAL WASTEWATER LAGOONS

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INTRODUCTION

Across Western Canada, lagoons are the most widely used method of treating municipal wastewater for smaller cities, towns, villages, hamlets, and resort communities. The successful operation of the lagoons requires that liquid effluent be stored on ground surface to depths of several metres. Traditionally these lagoons have been constructed with the soils native to the area, using a balanced cut and fill design for berms and the lagoon interior. Many of the lagoons in Western Canada constructed in this manner have been in service for over 35 years.

In the past seepage of more hazardous fluids stored in industrial waste or tailing ponds. In recent years, however, there has been increased public concern about the problem, and litigation arising from damage as a result of the lagoon seepage is becoming more frequent. The two main problems associated with lagoon seepage are:

1] Contamination of soil and ground water.
2] Addition of excess soil moisture creating salinity and waterlogging.

In response to growing public concern about the problem, all three Prairie provinces now have acts and regulations which pertain specifically to seepage from lagoons.

This paper will summarize the various acts and regulations in place in the Prairie Provinces, and will then discuss the design and construction considerations required to satisfy these requirements. A brief section at the end of the paper will specifically address the rehabilitation of existing lagoons. The intent of the paper is to provide an overview of the problem and available mitigative methods. The paper is not intended as a detailed technical design document.

EXISTING ACTS AND REGULATIONS PERTAINING TO WASTEWATER LAGOONS IN THE PRAIRIE PROVINCES

Government regulations across the Prairies are directed towards ensuring that municipal wastewaters are treated and disposed of in an environmental acceptable manner. Each province sets water quality standards and regulates the design and operation of wastewater treatment facilities to meet these standards while achieving maximum benefit from pollution control expenditures. The following sections summarize and briefly discuss the relevant regulations governing the design and construction of municipal wastewater lagoons in Alberta, Saskatchewan and Manitoba. A complete listing of the pertinent Acts and Regulations is appended to this paper.

In Alberta, the Department of the Environment administers wastewater lagoon regulations. Alberta regulations are put into effect through a series of guideline publications for municipal and other wastewater producers. One document in particular is very thorough in its treatment of seepage problems associated with lagoons. The 1983 publication "Design and Construction of Liners For Municipal Wastewater Stabilization Ponds" published by Alberta Environment sets out the Province's seepage standards. In Alberta, the seepage control criteria for these ponds specifies a maximum hydraulic conductivity, KL as defined by the following equation where L [m] is the thickness of the liner.

$$\text{Maximum } K_L \text{ (m/s)} = \frac{5.2 \times 10^{-9} \text{ m/s} \times L \text{ (m)}}{2 + L \text{ (m)}}$$

This approach is based upon an average wastewater depth of about 2 m, assumes atmospheric pore pressure below the liner and allows an arbitrary maximum seepage rate of 5.2 X 10 $^{-9}$ m/s. No allowances are made for local groundwater conditions in the design of the liner. This Alberta design standard complements the performance standards set out in the various Acts and Regulations regarding allowable limits of various substances in groundwaters and surface waters.

Alberta Environment reviews all applications for wastewater lagoon construction and requires varying degrees of investigation and design effort depending upon the amounts and types of waste to be treated and also upon the environmental sensitivity of the proposed lagoon site. As a minimum site investigation standard, Alberta Environment recommends anywhere from one borehole every two hectares, with a minimum of five per site to 1 borehole per hectare with a minimum of eight boreholes per site. All boreholes are recommended to extend at least 5 to 6 m below the lagoon invert elevation. Insitu testing of soil strength and hydraulic properties is encouraged to supplement laboratory testing programs.

Alberta appears to have lead the way in setting specific design and investigation criteria for wastewater lagoons, however, both Saskatchewan and Manitoba have similar requirements. Saskatchewan allows 15 cm per year (4 X 10^{-9} m/s) seepage from wastewater lagoons, although greater losses can be permitted if it can be demonstrated that surrounding land and groundwater will not be adversely affected. Manitoba has adopted a criteria for wastewater lagoons of having the interior surface of the lagoon underlain by at least one metre of soil having a hydraulic conductivity of 10^{-9} m/s or less. This 10^{-9} m/s criteria has been adopted from the regulations of the U.S. Environment Protection Agency. Alberta's 5.2 X 10^{-9} m/s standard follows Minnesota wastewater lagoon requirements.

Saskatchewan's sewage lagoon regulations are administered by the Saskatchewan Water Corporation. Saskatchewan Environment's 1986 publication "A Guide To Sewage Works Design" is the primary reference identifying items and factors that should be considered by sewage works designers and also providing accepted practices suitable for Saskatchewan conditions. Saskatchewan Environment also provides guidelines for investigating lagoon problems. The document "Investigative Protocol for Lagoon Problems" provides guidelines for the study of lagoon seepage concerns. This protocol also discusses soil quality degradation adjacent to lagoons.

The Saskatchewan guidelines specify effluent quality standards and recommend minimum siting setbacks from watercourses and development. The guidelines also stipulate a minimum of three test holes per site or one test hole for every two hectares, whichever is greater. All test holes are to be extended to at least 4 m below the lagoon floor elevation. Hydraulic conductivity tests are also required in Saskatchewan at the rate of one for every two hectares of lagoon. For insitu materials or soil liners, an on-site permeability of ten times any laboratory values is recommended to calculate seepage losses. Liners are less rigorously specified in Saskatchewan than in Alberta, with the Saskatchewan guidelines stating that where liners are used, the permeability, durability and integrity of any liner materials should be satisfactory for anticipated conditions.

In Manitoba, wastewater lagoon regulations are administered by the Department of Environment and Work Place Safety and Health. Manitoba also provides guidelines for sewage lagoons in the document "Design Objectives for Standard Sewage Lagoons". Manitoba's seepage criteria of 10^{-9} m/s has already been mentioned and, although the province has no specific site investigation requirements, they do require insitu field permeability tests of the constructed lagoon to ensure that minimum seepage standards are met prior to operation of the lagoon. Each wastewater lagoon is dealt with on a site-specific basis.

Experience elsewhere in the United States and Canada suggests that wastewater lagoon designers will tend to adopt the most stringent provincial regulatory requirements as a part of their normal practice. The lack of conflicts among the guidelines of the three Prairie provinces suggests that this will probably be the case in Western Canada

COMMON PROBLEMS ASSOCIATES WITH LAGOON SEEPAGE

Lagoon seepage has two main impacts on the environment surrounding the lagoon:

1) Contamination of soil and groundwater.
2) Addition of excess soil moisture creating salinity and waterlogging problems.

Contaminates contained in domestic wastewater can include nitrates, phosphorus, hydrocarbons, and various bacteria and viruses. The concern created by excessive lagoon seepage is that higher than allowable concentrations of these contaminants and other effluent constituents will render adjacent groundwaters and soils unsuitable for continued use. The magnitude of these potential contamination problems will depend upon the amount of seepage, the types and concentrations of contaminants, local hydrogeological conditions and proximity to population.

Soil salinity and waterlogging are two manifestations of the same problem - excess soil moisture. It is not completely correct to characterise soil salinity as only an excess soil moisture condition, in that excess soil moisture regimes do not always lead to soil salinity. The southern Prairies tend to have an excess of evaporation over precipitation, however, and the tendency will be towards a saline development.

SITE CHARACTERIZATION

Adequate characterization of existing or potential lagoon sites is essential if potential seepage problems are to be identified and the facilities properly designed to control the seepage. Site characterization is equally important for the rehabilitation of existing lagoons. Although many factors could be discussed when dealing with site characterization, three are considered of prime importance:

1. Regional Setting

The regional physiographic, geologic, topographic and environmental setting for the lagoon can generally be established from existing published information and a visual appraisal of the site. From this appraisal it should be possible to identify major underlying aquifers, the generalized soil profile, the regional direction of groundwater flow, the direction of surface water flow, proximity to major surface water bodies and any significant topographical features. Published information which is generally available in all three provinces includes:

- potentiometric surface maps
- 1:50000 contour maps
- 1:250000 contour maps

- regional geology and hydrogeology reports
- airphotos
- water well records
- existing geotechnical reports

A localized site investigation without tieing the site into the regional picture can often fail to identify long term seepage problems. In addition, the appraisal of the regional setting achieves the following objectives:

a] The soil profile and stratigraphy must be identified with confidence to a depth of 5 m below the lagoon floor.

b] Close attention should be paid to items such as stratification or varves in the soil strata, fractures or fissures in cohesive soil strata and discontinuities such as sand pockets or lenses. For wastewater lagoons a very accurate and descriptive field log is as important as the recovery and testing of high quality soil samples.

c] An adequate number of piezometers should be installed to determine the position of the groundwater table and the direction of flow.

3. Hydraulic Conductivity

All three Provinces have specific requirements in their acts and regulations with respect to hydraulic conductivity of the soil in the base and sidewalls of the lagoon. It is therefore, worth looking at the definition of hydraulic conductivity, which basically is a quantitative measurement of the ease of fluid flow through soil. The term is defined in the units of velocity, usually m/sec although in the past cm/sec has been used more extensively. The term should not be confused with "permeability" which is a measure of the porosity of the soil and is expressed in units of area [e.g. cm^2]. The term coefficient of permeability can be used interchangeably with hydraulic conductivity provided the fluid is water.

Typical hydraulic conductivity valves for various types of soils are listed in Table 1.

TABLE 1

Typical Hydraulic Conductivity valves
For Water Flow Through Soil
[After Terzaghi & Peck 1967]

Soil Type	Hydraulic Conductivity (m/sec)	Relative Permeability
Gravel	$> 10^{-3}$	high
Sandy Gravel	10^{-3} to 10^{-5}	medium
Sand	10^{-5} to 10^{-7}	low
Silty Clay	10^{-7} to 10^{-9}	very low
Clay	$< 10^{-9}$	impermeable

As discussed in a previous section all three Provinces require that the linings in lagoons have hydraulic conductivity in the rage of 1.0 to 5.2 X 10^{-9} m/s. Generally clays are considered to have hydraulic conductivities below 10^{-9} m/s and silts above 10^{-9} m/s. Most Prairie glacial clay tills fall in the range of 10^{-9} to 10^{-10} m/sec but this should be used with extreme caution as silt or sand tills will have substantially higher values. Although the three Province's requirements vary slightly, for ease of references in this paper, 1 X 10^{-9} will be assumed as the required hydraulic conductivity.

Methods of field and laboratory testing to determine hydraulic conductivity could be, and have been, the subject of entire text books. These methods will not be discussed here but several factors should be kept in mind.

a] Most laboratory tests measure the textural hydraulic conductivity of the soil only, and do not take into account stratification or fractures.

b] Laboratory tests on fine grained cohesive soils are difficult, expensive and time consuming to conduct.

c] Experienced personnel generally can estimate the textural hydraulic conductivity of soil within one order of magnitude on the basis of visual classification, Atterberg limits and grain size tests.

d] Hydraulic conductivity in the horizontal direction can be several orders of magnitudes higher than in the vertical direction.

e] Insitu hydraulic conductivity of a formation can be several orders of magnitude higher than the textural value due to soil structure and fracturing.

f] Field methods measure the mass hydraulic conductivity of a formation but must be conducted under closely controlled conditions if reliable values are to be obtained.

g] Clays compacted dry to optimum moisture may be hydraulic conductivity values several orders of magnitude higher than for the same clay compacted wet of optimum.

In the author's opinion, actual field or laboratory hydraulic conductivity tests are often not necessary for the satisfactory design of wastewater lagoons. They may be required where borderline soils such as silts or silty clays are encountered, or where the soils are severely stratified, in which case some type of field test is recommended.

Laboratory tests may also be necessary for recompacted soils in liners or berms. In may cases engineering fees could be better spent on more detailed field work and site characterization than on extensive laboratory tests for hydraulic conductivity. It should be emphasized that the foregoing approach to hydraulic conductivity may be suitable for lagoons but would not necessarily be satisfactory for ponds storing more hazardous wastes where better seepage control is required.

DESIGN CONSIDERATIONS

The nature of the seepage problem from wastewater lagoons is such that for virtually all cases the seepage can be adequately controlled by proper design and construction. This is not always the case with more hazardous industrial wastes where even very small losses can result in complex pump-back systems which must be operated throughout the life of the facility. Passive containment measures to prevent the problem of lagoon seepage are therefore, considered possible and preferable to ongoing measures to mitigate the effects of the problem.

Several excellent publications are available on the design of wastewater lagoons including the previously referred to "Design and Construction of Liners for Municipal Wastewater Stabilization Ponds" published by The Standards and Approvals Division of the Municipal Engineering Branch of Alberta Environment. The objective of this paper is not to reproduce in detail these design procedures but to summarize the considerations which should be taken into account in the design, to minimize the impact of seepage on the surrounding environment. The following general requirements should be assumed to apply to all lagoon construction:

1. Strip all organic soils from below the fills.
2. Compact clay soils in liners or berms to a moisture content which is at or above optimum moisture as defined by the Standard Proctor Compaction test (ASTM D698).

Special design considerations to control seepage will be presented for four typical situations.

1. Homogeneous Clays

This situation applies where the lagoons will be constructed entirely within a homogeneous clay with a hydraulic conductivity horizontally, vertically and in a recompacted state which is lower than 10^{-9} m/sec. For this case the clay formation itself provides the seepage control and no special design procedures are required other than the general requirements and good engineering practice.

2. Non Homogeneous or Fissured Clays

Many sites exist where the natural soils for the construction of the lagoon have textural hydraulic conductivity of less than 10^{-9} m/sec but where discontinuities, stratification or fractures make the insitu hydraulic conductivity much higher. At these locations the base of the lagoon and the side walls of the berms cut in native soil should be scarified, the moisture content of the soil adjusted and the material recompacted. The depth of scarification may vary depending on the severity of the site but generally 0.3 m has been used in the base of lagoons and 0.6 to 1.2 m in the sidewalls. Failure to undertake this relatively simple measure has often lead to serious seepage problems. Where more pervious sand pockets are found within a clay stratum at base elevation the sand should be excavated and replaced with the less permeable clay. A depth of cut of 0.6 m is commonly used in these circumstances.

3. Pervious Strata Overlying Impervious Clays

This is not an uncommon situation in Western Canada and one which can lead to serious damage problems outside the lagoon if not properly addressed. The reason for this is that if the seepage does occur, it will be trapped above the underlying impervious layer and will seriously affect surficial soil outside the lagoon. Surprisingly large areas outside the lagoon can become affected under these circumstances.

Fortunately this case can often be resolved without going to a lined pond. Two possibilities should be considered both of which require an on-site borrow source extending into the underlying cohesive soil. Depending upon the depth of the overlying sand and the surrounding topography this source can be inside or outside the lagoon.

a. If possible deepen the lagoon so that the floor is in the impervious layer. The clay from the base can then be used to line the sidewalls below original ground surface and to construct any berms that are required. This procedure may result in having to waste some of the overburden sand.

b. Install a compacted clay cutoff below the centreline of the berms extending 0.3 m into the underlying impervious layer. The cutoff should be as wide as normal construction equipment or about 3.0 m. Backhoes may be necessary where water is perched above the impervious layer. The cutoff should be backfilled with clay or cohesive soil up to the top of the berm. The more pervious soil can be used on the flanks of the berms.

4. **Deep Pervious Soils**

For conditions where the surficial soils have a textural hydraulic conductivity above 10^{-8} m/s and there is no underlying impervious layer within economical depth a liner is the most commonly used solution. Considerable design information is available for natural liners (clay), admixes to lower hydraulic conductivity of insitu soils, and synthetic liners. No attempt will be made to reproduce this information in this paper but a good reference is the previously referenced document on liners produced by Alberta Environment.

CONSTRUCTION CONSIDERATIONS

In the author's experience, for most of the cases where investigations have been carried out to determine the cause of excessive seepage from municipal wastewater lagoons, the problem has been related to poor construction practice as opposed to poor design. Admittedly, many of these are old facilities but the problems still occur all too frequently with recently constructed works. The most common construction problems that have been found are as follows:

1. Incomplete stripping of the topsoil below the berms resulting in a preferred seepage path between the native soil and the fill.

2. Mixing of topsoil with the fill in the berms.

3. The use of rejected pervious soils in the berms as opposed to the specified clayey soils. In other words, inadequate control of the fill going into the berms.

4. Where soil liners have been specified, the thickness has varied excessively and was non existent in some cases.

5. Compaction of clays at less than optimum moisture content.

6. Poor compaction.

On the surface these seem to be simple problems to rectify but in reality they are difficult to always achieve. Engineering fees spent on detailed hydraulic conductivity testing and liner design are rendered completely useless if the construction control is not to the same standard. The answer does not lie in specifying more tests per square metre or per site but in having the work in the field under the control of competent personnel to ensure that the design requirements are satisfied. More testing during or after construction cannot replace good sound engineering judgement during the actual construction process.

Rehabilitation of Existing Lagoons

Most of the preceding discussions apply to the rehabilitation of existing lagoons with a seepage problem as well as to proposed new facilities. The major difference is that the ground water regime around the existing lagoon will reflect the in-service condition as opposed to the original condition prior to construction. The higher groundwater levels and wetter soil conditions often limit the construction techniques which can be used to mitigate the problem. The following steps in the design process should be followed:

1. Site Characterization

   This is of equal or greater importance for a new lagoon than for an existing one. The assessment of the regional setting must determine the original natural site conditions as well as the present in-service condition.

2. As soon as the site characterization is complete the municipal designer must decide whether to:

   a) Rehabilitate the existing lagoon if it is adequately sized.

   b) Enclose and expand the lagoon by enclosing it in a new berm structure.

   c) Abandon the existing facility and build a new lagoon.

      Often the problems associated with keeping the present lagoon in service and construction in very wet soil conditions with a high water table make alternative (c) the most preferable.

3. If option a) or b) is selected, the limits imposed by the site conditions must be assessed to determine what alternatives are practically available for design. Complete dewatering and drying our of the base soils in a cell are a first step before a liner of any type can be attempted. Often a cutoff down to an impervious layer is impractical due to the high groundwater table. In one unique case it was successfully accomplished by winter construction when the surficial soils were frozen.

Any rehabilitation design is generally very site specific and requires very close co-operation between the owner, The municipal designer and the geotechnical consultant.

CONCLUSIONS

1. All 3 Prairie provinces have acts and regulations in place to control the design and construction of lagoons. Guideline publications are also available to assist in the design process itself.

2. The effect of seepage from municipal wastewater lagoons on the surrounding environment can generally be reduced to acceptable levels by reasonable design and construction practices for the lagoon itself. Only in unusual circumstances should it be necessary to have to design a facility for high seepage losses and to accommodate the effects of the seepage in the design.

3. Site characterization is an essential step in the design process and an understanding of the regional setting for the lagoon is of importance.

4. The factors affecting hydraulic conductivity must be well understood by the designer. Specifying minimum numbers of field or laboratory tests does not replace good sound engineering judgement in the design and construction process.

5. More lagoon seepage problems have been caused by poor construction practices than poor design. Good control of the construction is one of the most important aspects of achieving a satisfactory facility.

6. Rehabilitation of existing lagoons can be difficult due to the high water table, wet ground conditions and continuing service requirements. Early in the design process a decision on abandonment versus reconstruction must be made.

APPENDIX

Prairie Provinces Municipal Wastewater Lagoon Regulations and Acts

ALBERTA

Responsible Agency:

- Department of the Environment

Applicable Legislation

- The Clean Water Act
- The Water Resources Act
- The Groundwater Control Act
- The Land Surface Conservation & Reclamation Act
- Public Health Act

Design Guidelines:

- Recommended Standards for Water Supply & Sewerage
- Waste Water Effluent Guidelines for Alberta Fertilizer Plants
- Dam and Canal Safety Guidelines, Alberta Environment, Publication D.S.3., 1979
- Inspection of Small Dams, Alberta Environment, Publication D.S.5., N.D.
- Design and Construction of Liners for Municpal Wastewater Stabilization Ponds, Alberta Environment, 1983

MANITOBA

Responsible Agency:

- Department of Environment and Work Place Safety and Health

Applicable Legislation:

- Clean Environment Act
- Public Health Act
- Dyking Authority Act
- Rivers & Streams Act
- Water Rights Act
- Conservation Districts Act
- Water Supply Districts Act
- Regional and Municipal Planning Regulations

Design Guidelines:

- Design Objectives For Standard Sewage Lagoons, Environmental Management, July, 1985
- Addendum to "Design Objectives For Standard Sewage Lagoons" Respecting Seepage, July, 1985

SASKATCHEWAN

Responsible Agency:

- Saskatchewan Water Corporation

Applicable Legislation:

- The Saskatchewan Water Corporation Act
- The Environmental Management and Protection Act
- The Water Pollution Control and Waterworks Regulations

Design Guidelines:

- A Guide to Sewage Works Design, Saskatchewan Environment, March, 1986
- Water Quality Objectives
- Sewage Effluent Irrigation Guidelines
- "Sewage Lagoon Problems - Investigative Protocol Study", Saskatchewan Environment, November, 1984

COAGULATION METHODS OF ALUMINUM SALTS

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INTRODUCTION

Surface and ground water consist of heterogeneous mixtures of substances of varying sizes. These include dissolved, colloidal and suspended materials which may be living or nonliving, natural or synthetic, beneficial or harmful, coloured or colorless, toxic or nontoxic, organic or inorganic, biodegradable or non-biodegradable. For years undesirable substances in water have been removed through processes known as coagulation/flocculation followed by sedimentation and/or filtration.

Coagulation/flocculation in simple terms is the process whereby dissolved, collodial or suspended particles are brought together with the aid of chemicals to form larger diameter aggregates that can be settled out or filtered out of the liquid phase.

The terms dissolved, colloidal and suspended are defined according to their particle size (1). Dissolved substances become part of the solution and can not be filtered out. Suspended substances are large particles (>100 microns) which can be seen with the human eye. These particles can be filtered out or settle out on their own. Colloidal materials fall between these two classes. Colloidal substances range in size from one nanometer (10^{-9} m) to one micron (10^{-6} m) in at least one dimension. These particles are too fine to settle out of the suspension by gravity and would require membrane filtration if they were not coagulated and removed as larger masses.

The undesirable substances removed in potable water treatment generally include turbidity and colour (2). Turbidity refers to suspended and colloidal particles which scatter light causing a cloudy appearance. Colour is caused by dissolved and colloidal substances that absorb light. Suspended particles may be organic or inorganic and are usually solids. Most often it is the naturally produced organic matter called humic substances, i.e. humic acid, fulvic acid and humins, and metallic ions (such as iron and manganese) that cause a yellow or brown colour in waters (3). Both turbidity and colour are aesthetic problems and may be associated with health hazards; therefore, removal is important.

Coagulation is a process prominent not only in raw water treatment but also in wastewater clarification, papermaking, sludge dewatering, and mineral processing.

In this paper emphasis will be placed on the coagulation chemistry of aluminum salts, in particular alum, in water treatment.

COAGULATION VS. FLOCCULATION

There has always been confusion associated with the use of the two terms coagulation and flocculation. Recent work has attempted to differentiate these terms as two distinct processes.

Most colloidal particle or molecular surfaces in water show a net negative surface charge. The negative charge on their surfaces cause them to repel neighboring particles stabilizing themselves in suspension. Most inorganic salts such as aluminum sulphate, yield various cationic species when dissolved in water. The positively charged coagulant and negatively charged particles are then electrostatically attracted.

Coagulation is defined as the process whereby the forces holding the solids in suspension are overcome or neutralized (4). Coagulation is also referred to as a destabilization of suspended particles. This process is usually best accomplished by adding chemical coagulants and by applying intense mixing conditions.

Flocculation is the process whereby destabilized suspended solids are brought into contact to form larger aggregates called flocs (4).

Coagulation and flocculation of contaminants by metal coagulants has been described by several researchers (1,5) to be very complex on a molecular level, involving charge neutralization, compression of the electrical double layer, adsorption of soluble contaminants on the solid coagulant, stoichiometric precipitation, bridging, and enmeshment of colloidal particles in a voluminous floc.

Work in the literature on coagulation has been reviewed and has revealed that most of the coagulation mechanisms have been derived using aluminum sulphate because of its extensive use in the industry. One of the most significant contributions to the understanding of the coagulation process has been the elucidation of how aluminum behaves in aqueous solution. It has been shown that the formation of aluminum hydroxide is not the only critical reaction occurring in coagulation. The chemistry is far more complicated due to aluminum hydrolysis species.

AQUEOUS CHEMISTRY OF ALUMINUM

Most of the studies into aqueous aluminum chemistry have used the sulphate salt as the source of aluminum. Hence, due to the lack of research on coagulation with other aluminum salts, alum will be discussed here in detail. The other sources of aluminum such as sodium aluminate, aluminum chloride or polyaluminum chloride need further investigation before an in-depth comparison can be done.

Although alum has been used extensively, the fundamental knowledge of alum's interactions with other materials is inadequate. The complexity of the chemistry of alum in solution has rendered the study of these processes very difficult. A description of the wide variety of species of aluminum will be discussed illustrating the complexity of aluminum.

For many years it was thought that alum, when dissolved in water, produced only the unhydrated aluminum (Al^{3+}) ion which did all the work in coagulation. It was believed that this ion coagulated the negatively charged particles in solution by purely electrostatic interaction. The addition of soda ash produced a floc of aluminum hydroxide, $Al(OH)_3$, which entrapped the fine neutralized particles and removed them. Although there still remains some basis for these two ideas, they have undergone considerable revision in the past few years (6).

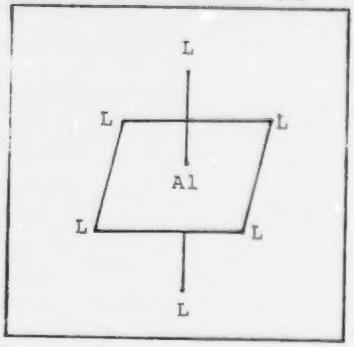
Dissociation of Alum

Solid aluminum sulphate is considered to be completely ionized, with the hydrated aluminum cations and sulphate anions separated by the short distances which prevail in the crystal lattice. (See equation 1 of Table 1).

Since the common coordination number of aluminum is six, each cation in the crystal holds or coordinates six molecules of water. The remaining water is believed to be distributed around the anions in the lattice (7).

The hydrated aluminum ion with six water molecules surrounding it forms an octahedron as shown in Figure 1, where L is a water molecule. This structure forms the inner coordination sphere.

FIGURE 1
Hydrated Aluminum Ion (8)



Hydrolysis

Following almost instantaneous dissociation of alum in an aqueous medium the hydrated aluminum ion is capable of undergoing several reactions. At pH <3 the simple trivalent, hydrated cation is the sole aluminum species present in solution. Above this pH, the replacement of a water ligand with a hydroxide group (OH^-) in the inner coordination sphere starts to become significant. This is referred to as hydrolysis. (See equations 2-5 in Table 1).

TABLE 1
Chemical Species of the Aluminum Sulphate Water System (7,8,9)

Dissociation

$$[Al_2(SO_4)_3.14H_2O] + H_2O \longrightarrow 2[Al(H_2O)_6]^{+3} + 3SO_4^{-2} + 2H_2O \qquad (1)$$

Hydrolysis

$$[Al(H_2O)_6]^{+3} + H_2O \longrightarrow [Al(OH)(H_2O)_5]^{+2} + H_3O^{+1} \qquad (2)$$

$$[Al(OH)(H_2O)_5]^{+2} + H_2O \longrightarrow [Al(OH)_2(H_2O)_4]^{+1} + H_3O^{+1} \qquad (3)$$

$$[Al(OH)_2(H_2O)_4]^{+1} + H_2O \longrightarrow [Al(OH)_3(H_2O)_3]^0 + H_3O^{+1} \qquad (4)$$

$$[Al(OH)_3(H_2O)_3]^0 + H_2O \longrightarrow [Al(OH)_4(H_2O)_2]^{-1} + H_3O^{+1} \qquad (5)$$

Inactivation by Anions

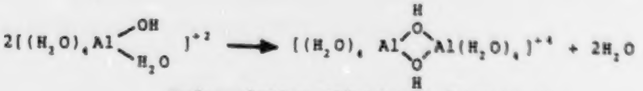
$$[Al(H_2O)_6]^{+3} + SO_4^{-2} \longrightarrow [Al(SO_4)(H_2O)_4]^{+1} + 2H_2O \qquad (6)$$

$$[Al(OH)(H_2O)_5]^{+2} + SO_4^{-2} \longrightarrow [Al(SO_4)(H_2O)_4]^{+1} + OH^{-1} + H_2O \qquad (7)$$

Polymerization (Olation)

$$2[Al(OH)_2(H_2O)_4]^{+1} \longrightarrow [Al_2(OH)_4(H_2O)_6]^{+2} + 2H_2O \qquad (8)$$

$$[Al_2(OH)_4(H_2O)_6]^{+2} + [Al(OH)_2(H_2O)_4]^{+1} \longrightarrow [Al_3(OH)_6(H_2O)_8]^{+3} + 2\,H_2O \qquad (9)$$



$$2[(H_2O)_4Al(OH)(H_2O)]^{+2} \longrightarrow [(H_2O)_4\,Al(OH)_2Al(H_2O)_4]^{+4} + 2H_2O \qquad (10)$$

Hydroxylation (Aluminate Formation)

$$[Al(OH)_3(H_2O)_3] + OH^{-1} \longrightarrow [Al(OH)_4(H_2O)_2]^{-1} + H_2O \qquad (11)$$

$$[Al(OH)_4(H_2O)_2]^{-1} \longrightarrow (AlO_2)^{-1} + 4H_2O \qquad (12)$$

By examining Figure 1, L can now represent either OH^- or H_2O in the structure. These hydrolysis products can be termed as hydrated hydroxyaluminum ions. These reactions, where hydrating water molecules are being displaced, occur very rapidly (0.01 to 1 second) and hence intense mixing conditions are recommended for complete development of species (8).

The equilibrium constant for the first hydrolysis reaction has been accurately determined by several investigators employing a variety of measurement techniques (7). Equilibrium constants for succeeding hydrolysis steps are not well established. Thus Table 1 should be considered only a simplified summary of the possible species formed. With each successive loss of water molecule the hydroxyl ion content of the solution decreases as the hydronium (H_3O^{+1}) ion or hydrogen ion concentration increases.

It is the hydrolysis species of aluminum that are said to be most active during coagulation.

Inactivation

Sulphate ions also may coordinate with aluminum, replacing coordinated water molecules and/or hydroxide groups. The aluminum cations containing sulphate are said to be "inactivated" or unavailable for coagulation reactions. The structure can be represented where L is either OH^-, H_2O or SO_4^{-2} (Figure 1). The aluminum-sulphate complexes formed are relatively stable and do not show the normal behaviour of either the aluminum ion or the sulphate ion (7). (See equations 6 & 7 in Table 1). Since complete dissociation occurs rapidly upon dissolution of alum this reformation of aluminum-sulphate complexes is not believed to occur to any great extent, inactivating only very small amounts of aluminum from participating in coagulation.

Coordination with Anions

Aluminum cations have a general tendency to coordinate with simple anions replacing the hydrated water molecules in Figure 1. The coordinated complexes formed by aluminum show greater stability when the anions of weak acids are involved. Aluminum coordinates very strongly with weak acid anions such as hydroxide and carboxylate ions, moderately with sulphate, and weakly or not at all with chloride and nitrate ions (7). Hence, hydrolysis reactions, where water molecules are replaced with hydroxyl groups, are favoured over inactivation reactions involving sulphate.

Polymerization or Olation

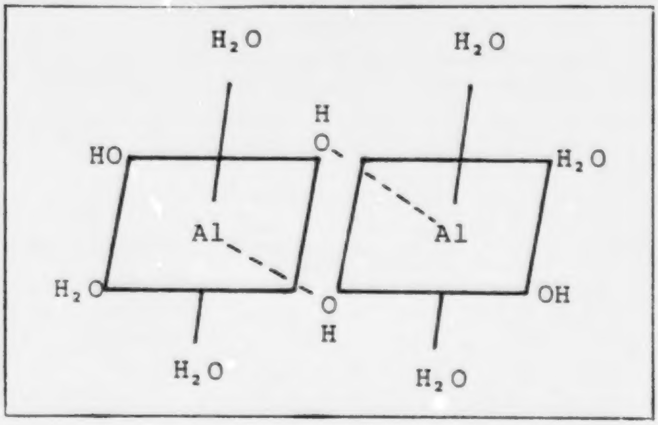
It is possible to build polymers from several of the hydrolysis products as in equations 8-10.

Hydrated hydroxyaluminum ions produced by partial hydrolysis can dimerize and polymerize by a process known as "olation" (7). One aluminum atom coordinates with a hydroxyl group held tightly by another aluminum atom, and vice versa. Each aluminum gives up a molecule of coordinately bound water. This leads to the complex form shown in Figure 2.

The interaction of aluminum ions with hydroxyl ions can lead to a variety of semi-polymeric products by this hydroxyl bridging (olation) process. Many cationic polymeric species, containing as many as 13 aluminum atoms, have been suggested to exist (7). These complexes are often long, linear molecules (8). The species most often proposed is $Al_8(OH)_{20}^{+4}$.

FIGURE 2

Example of Olation Bonding (7)



These reactive species required for coagulation are best developed under intense mixing conditions. Other physical conditions also affect the formation of these aluminum species such as temperature, concentration and pH. Generally as the temperature of the water decreases, it takes longer for full development of all the species (10). Most species develop in the typical pH range encountered in water treatment of pH 4.5 to 6.0. Polymerization increases at higher pH values, in more concentrated solutions, and at higher temperatures.

Hydroxylation (Aluminum Formation)

Aluminum hydroxide, $Al(OH)_3$, a product of the hydrolysis reactions, is very insoluble in water. (See Equation 4 of Table 1). Aluminum hydroxide takes 1 to 7 seconds to form which is longer than other hydrolysis products due to the number of intermediates formed prior to precipitation of the hydroxide (9). Aluminum hydroxide can be redissolved in water by strong alkali; the product is aluminum tetrahydroxide (equation 9), a monovalent anion which is water-soluble at pH 10 and above. In one sense, this product is merely the fourth step in the aluminum hydrolysis or hydroxylation process, as seen by comparing equations 2-5 with 9. In another sense aluminum tetrahydroxide is also the fully hydrated anion of sodium aluminate, $NaAlO_2$, as indicated in equations 11 and 12. Sodium aluminate is another commercially-available material used to introduce aluminum into the systems.

COAGULATION MECHANISMS

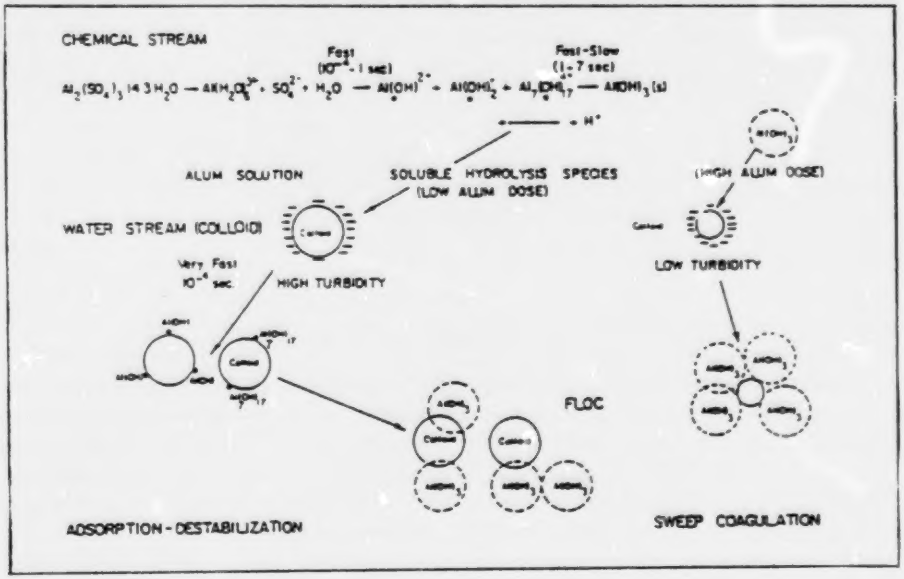
The aqueous chemistry of aluminum has been shown to be complex because of the numerous hydrolysis intermediates formed in addition to the precipitation of aluminum hydroxide. Extensive work has shown that the hydrolysis species of aluminum (III) and aluminum hydroxide are active during coagulation. The hydrolysis species and hydroxide precipitate interact with colloidal substances by two separate mechanisms.

Coagulation of colloids by aluminum occurs predominantly by two mechanisms: (1) adsorption of positively charged soluble-hydrolysis species on negatively charged colloids causing charge neutralization (or destabilization) and (2) sweep coagulation where the colloid is entrapped within the precipitating aluminum hydroxide (9,11).

Figure 3 illustrates the two coagulation mechanisms termed adsorption-destabilization (by aluminum hydrolysis species) and sweep coagulation (with aluminum hydroxide).

FIGURE 3

Reaction Schematics of Coagulation (8)


CHEMICAL STREAM
Fast
(10⁻⁴ - 1 sec)
Fast-Slow
(1 - 7 sec)
ALUM SOLUTION
SOLUBLE HYDROLYSIS SPECIES
(LOW ALUM DOSE)
(HIGH ALUM DOSE)
WATER STREAM (COLLOID)
Very Fast
10⁻⁴ sec
HIGH TURBIDITY
LOW TURBIDITY
FLOC
ADSORPTION - DESTABILIZATION
SWEEP COAGULATION

Before discussing the mechanisms in coagulation of colloids the complexity of the entire system must be realized. Raw water is a complex system with a mixture of positively charged and negatively charged particles and ions. It is the purpose of coagulation to destabilize a suspension of negatively charged colloidal particles and form large masses which will settle out from the raw water.

Upon addition of a chemical coagulant several chemical species, positively and negatively charged species, are introduced. Not only are these products involved in coagulation reactions with colloids but also with competing reactions with other anions present. These other anions are referred to as background ions and vary for each water source (i.e. sulphate, phosphate, etc.).

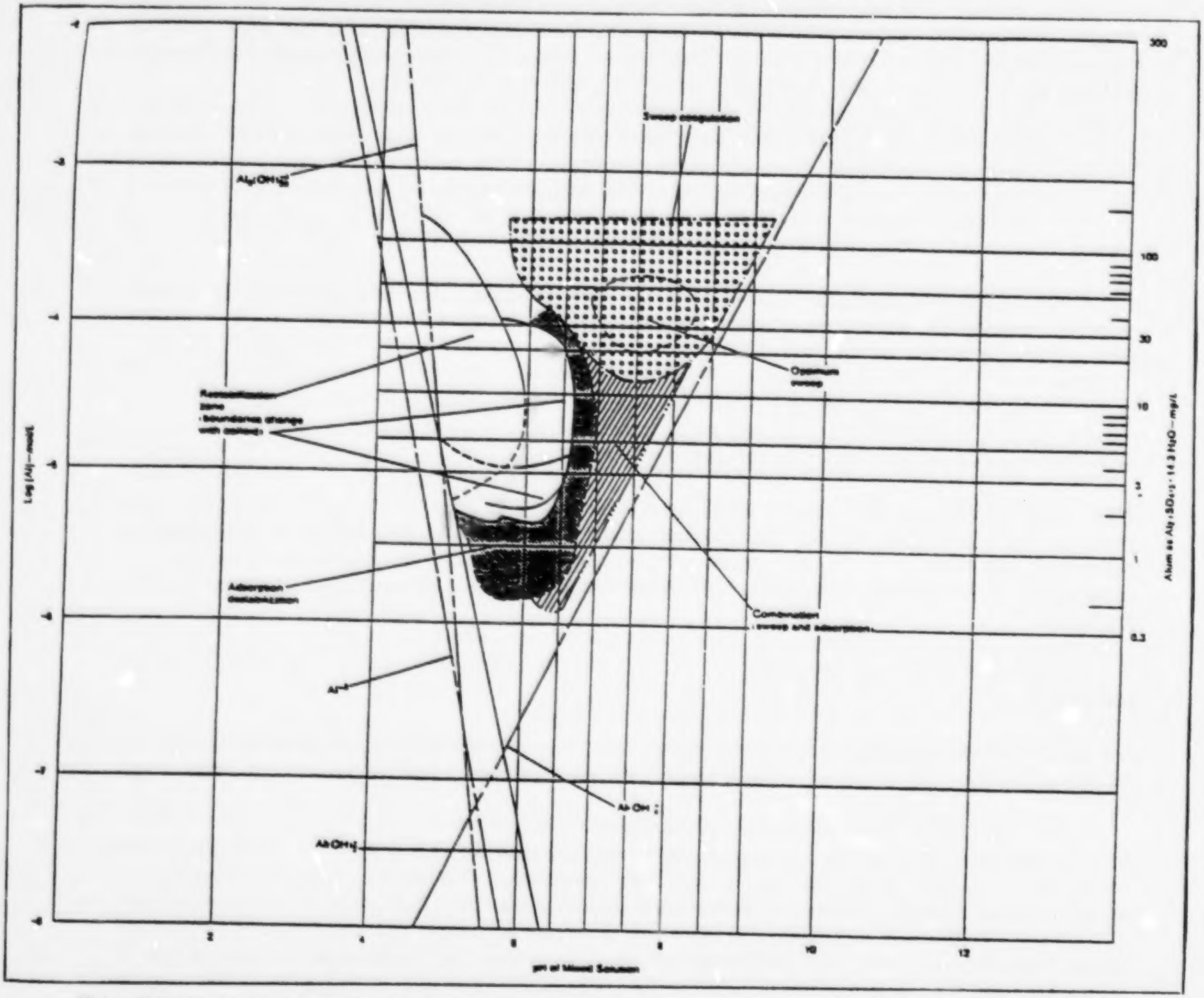
The goal is to add the proper alum dose to yield sufficient amounts of hydrolysis products and aluminum hydroxide to coagulate the colloids and satisfy competing reactions without overdosing the chemical coagulant. When overdosing of alum occurs, the effectiveness of the coagulant is substantially decreased causing a restabilization of the suspension. This restabilization is caused when an excess of positive charges repel each other, thus reforming a suspension.

Several researchers (9-21) have examined the mechanisms involved in coagulation. Amirtharajah and Mills' (9) alum coagulation diagram, shown as Figure 4, helps to bridge the knowledge from the theoretical mechanisms to practical application in the water treatment plant. This diagram has provided a predictive tool to define coagulant dosage and pH conditions where each of these mechanisms would dominate and provide the most efficient coagulation. Figure 5 shows the very specific areas where coagulation would occur and also the major mechanism causing coagulation.

Using the diagram it is possible to predict specific alum dosages and the pH where optimum coagulation would occur. Analysis of specific regions of Figure 4 can help to interpret the oftentimes confusing data generated from jar tests.

FIGURE 4

Design and Operation Diagram for Alum Coagulation (9)



This diagram is based on the findings that alum coagulation depends primarily on the final pH of the water and the concentration of alum dosage. Several researchers work have shown many similar results when numerous different turbidity conditions existed (9).

Figure 4 plots alum dosages as log(Al) in moles per litre (mol/L) and as milligrams per litre (mg/L) against pH of the mixed solutions. The solid straight lines represent thermodynamic stability lines for hydrolysis species. The areas between these lines indicates the major mechanism that dominates.

On the left hand side of the diagram are the stability lines for the positively charged hydrolysis products namely, Al^{+3}, $Al_8(OH)_{20}{}^{+4}$ and $Al(OH)_2{}^{+1}$. These hydrolysis species interact with the negatively charged colloids by the adsorption-destabilization mechanism. Not only does neutralization occur near these bounderies but also restabilization when an alum overdose occurs. Restabilization occurs when all negative particles have been charge neutralized and the excess positive charge causes repulsion. Hence, a suspension is reformed.

On the right hand side of the diagram is the formation of the negatively charged species, $Al(OH)_4{}^-$. Aluminum hydroxide is precipitated primarily on the right side of the diagram where the pH is 6 or greater and the alum dosage is 10 mg/L or above. Soluble hydrolysis products active in the adsorption-destabilization mechanism are dominant at lower pH conditions and alum doses.

The area for optimum sweep coagulation, or the area for best settling floc with the lowest coagulant dosages, is specifically defined by the major parameters of alum dose of 20 to 50 mg/L with a final pH of 6.8 to 8.2 (9).

It has been observed that destabilization works well with waters of high turbidity and high organic colour content (TOC) whereas sweep coagulation has been observed to work well with waters of low turbidity and low colour (9).

Coagulation conditions utilized by each specific plant will fall within various areas of the graph at different times of the year as raw water quality continually changes. This diagram, along with the results of the jar tests, will assist the operator in knowing which variable to adjust, pH or alum dose. Coagulation is undoubtedly a complex but necessary process in water treatment.

CONCLUSIONS

The processes of coagulation and flocculation have been differentiated. Coagulation is defined as charge neutralization and flocculation is the aggregation of smaller particles into larger flocs.

Aqueous chemistry of aluminum has been shown to be very complex with numerous hydrolysis species formed prior to the precipitation of aluminum hydroxide. Hydrolysis ions and polymers are active in coagulation as well as aluminum hydroxide.

Coagulation in water treatment with aluminum salts occurs by two mechanisms: (1) adsorption of hydrolysis species on the colloid (charge neutralization) and (2) entrapment of the colloid within the precipitating aluminum hydroxide (sweep coagulation).

The alum coagulation diagram developed by Amirtharajah and Mills (9) can be used for practical applications in effective operation of alum coagulation, determining optimum coagulant dose and pH conditions.

A better understanding of colloid chemistry and chemical hydrolysis of aluminum sulphate has enabled an interpretation of the mechanism of coagulation. This knowledge, combined with jar test results, offers the operation the tools to better control and optimize his/her water treatment operation.

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HIGH QUALITY WATER TREATMENT SYSTEM
FOR THE FRESHWATER INSTITUTE, WINNIPEG, MANITOBA

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Reid Crowther & Partners Ltd.

E. Scherer
Freshwater Institute

R. Wagemann
Freshwater Institute

BACKGROUND

The Freshwater Institute is a Federal Fisheries and Oceans research facility located in the City of Winnipeg on the University of Manitoba campus. The Institute is the headquarters for some 300 scientists and supporting technical and administrative personnel whose work is primarily devoted to research and monitoring activities in aquatic ecology, toxicology, and related fields. In order to perform this work, a reliable supply of high quality non-toxic water is required for holding and culturing live fish and other aquatic species, and as dilution water for research experiments.

The Institute receives its water supply from the City of Winnipeg's water distribution system. The raw water source for the City is Shoal Lake located near the Manitoba/Ontario boundary. The water is conveyed in a 140 km aqueduct to Winnipeg where it is stored in reservoirs prior to treatment and pumping to the distribution system. The treatment provided by the City consists of chlorination and fluoridation. In the past when the water storage reservoirs were uncovered, copper salts have been added to the reservoirs on occasion to control algae blooms occurring therein.

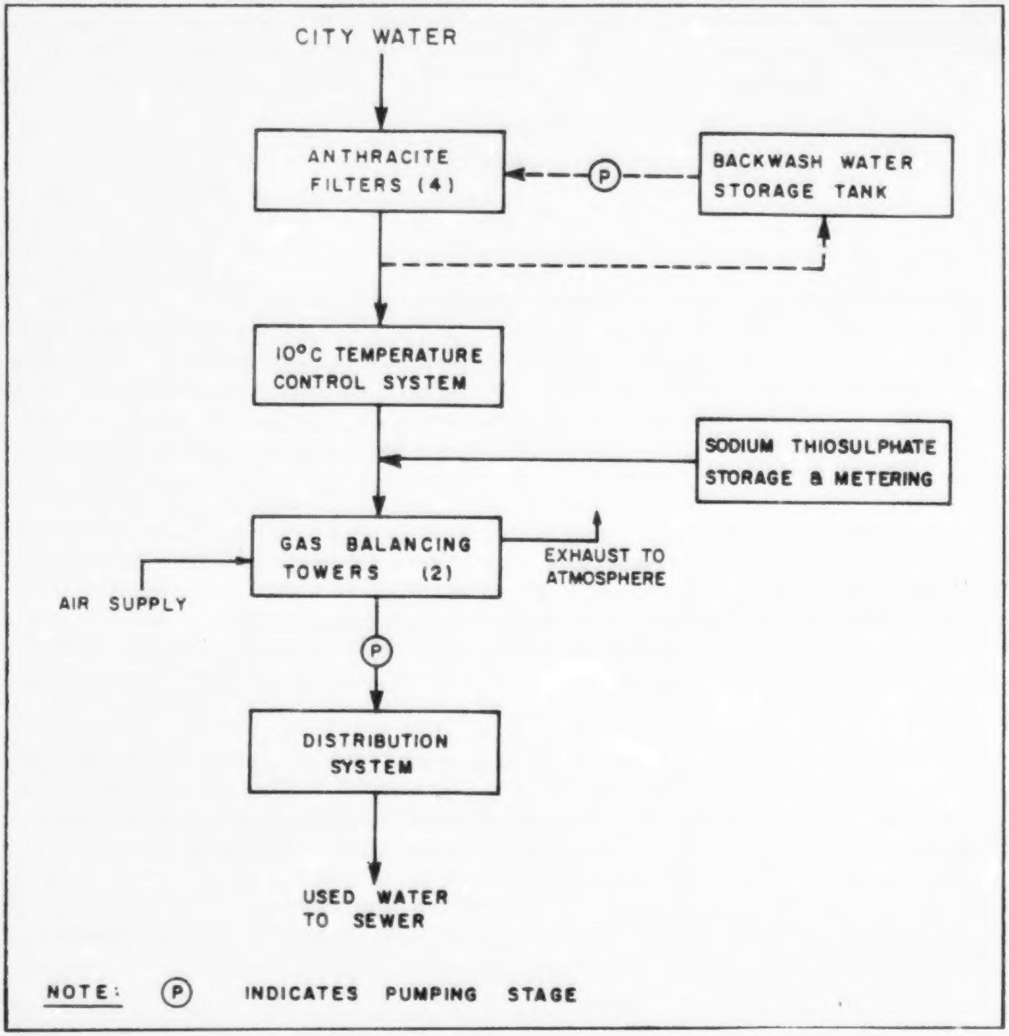
The chlorine residual in the City's tap water is acutely toxic to aquatic organisms even at relatively low concentrations. Therefore the chlorine must be removed to make the tap water acceptable as a medium in which the organisms can survive in a healthy state in the laboratory. In addition, fluctuating concentrations of metals and other trace contaminants that occur in the City's water are undesirable when conducting sensitive bioassay work or doing long term experiments at low concentration levels. Furthermore the wide seasonable fluctuations in water temperature (from approximately 2.5°C in water to over 20°C in summer) are not acceptable for controlled experimental work on aquatic species.

ORIGINAL TREATMENT SYSTEM

When the main Freshwater Institute building was completed in 1972, a treatment system consisting of the components sketched in Figure 1 was installed by the architect. The system incorporated anthracite pressure filters for particulate removal, a 10°C water temperature control system, sodium thiosulphate addition for chlorine residual removal, gas balancing towers for producing a product water saturated with air at 10°C, and a dechlorinated water distribution system.

FIGURE 1

Block Diagram of Original Water Treatment System
Freshwater Institute, Winnipeg, Manitoba


CITY WATER
ANTHRACITE FILTERS (4)
P
BACKWASH WATER STORAGE TANK
10°C TEMPERATURE CONTROL SYSTEM
SODIUM THIOSULPHATE STORAGE & METERING
GAS BALANCING TOWERS (2)
EXHAUST TO ATMOSPHERE
AIR SUPPLY
P
DISTRIBUTION SYSTEM
USED WATER TO SEWER
NOTE: P INDICATES PUMPING STAGE

The thiosulphate in the product water caused interferences with metal toxicity studies and, in addition, a marked deterioration in product water quality was experienced over the first few years of operation of the system. Of particular concern were intermittent "slugs" of particulate matter that occurred in the product water at frequent intervals.

A systematic investigation of the original treatment system was conducted in 1978/79 and resulted in the following conclusions:

- The filter backwash pump capacity only provided about 25% of the required flowrate to achieve a conventional backwashing rate commonly used to clean the filter effectively.

- Due to the configuration and dimensions of the filter vessels (cylindrical tanks mounted with their long axes in the horizontal plane), it was not possible to backwash the filters at conventional backwash rates without losing filter media.

- Because of the low backwashing rates, the filter beds were not being cleaned properly during the backwash cycle. Consequently the filter media became "packed" with dirt in zones not directly over the underdrain strainers and the air scouring lines.

- This "packing" effect led to short-circuiting or channelling of raw water through the filters. Evidence of this was confirmed when zooplankton and algae species originally presumed present in the Shoal Lake surface water supply were found in the product water of the original treatment system in significant numbers.

- The "slugs" of particulate matter were attributed to biological slime growths consisting of the sulphur-oxidizing filamentous bacteria *Beggiatoa* as the primary colonizer. Once the *Beggiatoa* rotifers, amoebae, and oligiochates occurred. The *Beggiatoa* were linked to the addition of sodium thiosulphate used as a dechlorinating agent.

It was decided to proceed with the design of a new treatment system incorporating new filters and chlorine removal technology that did not require the addition of thiosalts.

WATER QUALITY AND QUALITY REQUIREMENTS

The scientists at the Freshwater Institute participated in a survey to identify dechlorinated water demands. The survey results indicated the following potential water demands:

Median demand = 550 L/min
Peak demand = 925 L/min

at a product water temperature of 10°C. In addition, it was specified that up to half of the product water should be deliverable at 5°C to facilitate low temperature research work with Arctic species.

The scientists also were requested to assemble a list of water quality requirements to use as "target" values for the product water of the new treatment system. Accordingly, a number of physiologists and toxicologists at the Institute reviewed the scientific literature (APHA 1980; ASTM 1975, 1978, 1982; Wedemeyer and Yasutake 1977; NAS 1972; McKee and Wolf 1963) and developed a list of maximum desirable concentrations of chemical compounds in the product water of the new treatment system. The numbers on this list were termed "target" values and these are presented in the first column of numbers in Table 1.

TABLE 1

Water Quality Target Values, Anticipated Water Quality, and Actual Product and City Water Quality

Parameters	Units	FWI Target Levels	Anticipated Product Water Quality	Actual Product Water Quality*	City Water Quality at FWI*
pH	(none)	7 to 8	7.4	7.86 - 8.22	7.44 - 8.24
Hardness	(mg/L CaCO$_3$)	Shoal Lake (~80)	~80	-	-
Alkalinity	(mg/L CaCO$_3$)	Shoal Lake (~75)	~75	-	-
Conductivity	(umho/cm)	Shoal Lake (~165)	~165	-	-
Total Suspended Solids	(mg/L),(u)	<80, <10	<5, <10	0.5 - 1.0	0.5 - 2.0
Residual Chlorine	(ug/L)	<3	10 to 15	0 - 5	100 - 640
T.O.C.	(mg/L)	<2	-	-	-
C.O.D.	(mg/L)	<5	-	-	-
Ammonia (unionized)	(ug/L)	<5	<5	0.1 - 1.3	0.1 - 0.8
Nitrite	(ug/L NO$_2$)	<20	1	<0.5 - 8.0	<0.5 - 1.0
Nitrate	(ug/L NO$_3$)	<10	<10	1 - 105	1 - 62
Aluminum	(ug/L)	<1	<5	2 - 13**	5 - 22**
Arsenic	(ug/L)	<1	1	-	-
Cobalt	(ug/L)	<1	<2	-	-
Chromium	(ug/L)	<1	-	-	-
Copper	(ug/L)	<1	<3	0.2 - 1.0	0.2 - 1.1
Iron	(ug/L)	<1	<10	30 - 80	35 - 180
Manganese	(ug/L)	--	2	0.5 - 14*	2 - 26**
Lead	(ug/L)	<1	<1	-	-
Selenium	(ug/L)	--	<1	-	-
Nickel	(ug/L)	<1	<2	-	-
Zinc	(ug/L)	>1	<1	<0.05 - 0.7	<0.05 - 0.4
Cadmium	(ug/L)	<0.1	0.1	-	-
Mercury	(ug/L)	<0.1	-	-	-
Silver	(ug/L)	<0.1	-	-	-
Boron	(ug/L)	<100	-	-	-
Fluoride	(ug/L)	<100	920	-	-
Total Organophosphorus Pesticides	(ng/L)	<50	<10	-	-
Total Organochlorine Pesticides + PCB's	(ng/L)	<50	<50	-	-
Total HC's C<10 (Aromatics & Aliphatics)	(ug/L)	--	27	-	-
CCl$_4$	(ug/L)	--	Trace	-	-
CHCl$_3$	(ug/L)	--	<20	-	-
CHCl$_2$Br	(ug/L)	--	<20	-	-

Notes: - indicates no data available
-- indicates no target value specified to predict anticipated product water quality
* From Wagemann et al (1987a and 1987b) data for April 1986 to September 1986
** Dissolved values

TREATMENT SYSTEM CONCEPT

The purpose of the treatment system is to remove the chlorine residual, as well as particulate and organic matter from Winnipeg tap water, and to provide a high quality disinfected and air-saturated product water at both 10°C and 5°C on a 24-hour continuous basis for use by the scientists at the Institute.

With regard to product water quantity, the design flowrate for 10°C water was selected to provide an approximate 10 percent margin of safety above the median demand and, in addition, sufficient product water storage capacity would be provided to meet the peak demand superimposed on the design flowrate for a 1 hour duration. Accordingly, the design flowrate and storage capacity are:

10°C product water flowrate = 600 L/min
10°C product water storage = 20,000 L

Likewise for the 5°C product water, the design flowrate and storage capacity are:

5°C product water flowrate = 300 L/min
5°C product water storage = 10,000 L

The 5°C product water volumes are drawn at the expense of water delivered to the 10°C users. This arrangement is coordinated and monitored by the scientists.

Sufficient redundancy is incorporated into this system to allow for the production of 600 L/min of product water despite a shutdown of any major piece of equipment either for maintenance or in case of breakdown. Furthermore, critical components of the system are tied into the standby power generating facilities at the Institute so that the product water supply can be available on a continuous basis and long term experiments are not aborted.

With respect to product water quality, feasibility studies indicated that an affordable system that would meet all the target values listed in Table 1 was not available and that sacrifices would be necessary particularly for parameters such as fluoride and some of the metals, notably aluminum and iron. Consequently a state-of-the-art system employing the following unit operations was designed and built:

- Mixed media filtration
- Ozonation
- Activated carbon adsorption
- Ultraviolet sterilization
- Dissolved gas balancing
- Micro-filtration

A block diagram schematic illustrating the treatment system concept is shown in Figure 2. The basic design parameters of the major system components are listed in Tables 2 to 8 inclusive. Not indicated in Figure 2 are the elements of flow and temperature control in the treatment system. The flow of water through the system is controlled as follows:

- User demand is sensed by a water level sensing device in the product water storage tanks. This device is tied to a rate-of-flow control valve which admits more or less water through the gas balancing towers as required.

- A change in flowrate at the gas balancing towers is reflected upstream in the treatment system train by a change in level in the ozone reactor vessels. This is detected by a level sensing device which is tied to a rate-of-flow control valve that admits more or less water into the ozone reactor vessels to maintain a constant water level therein.

- The treated water supply both the 10°C and 5°C distribution systems is maintained fresh by return lines from the ends of the distribution systems to the ozone reactor vessels. The flowrates in these return lines are set by a series of manual flow control valves with rotameters to give a visual indication of flowrate.

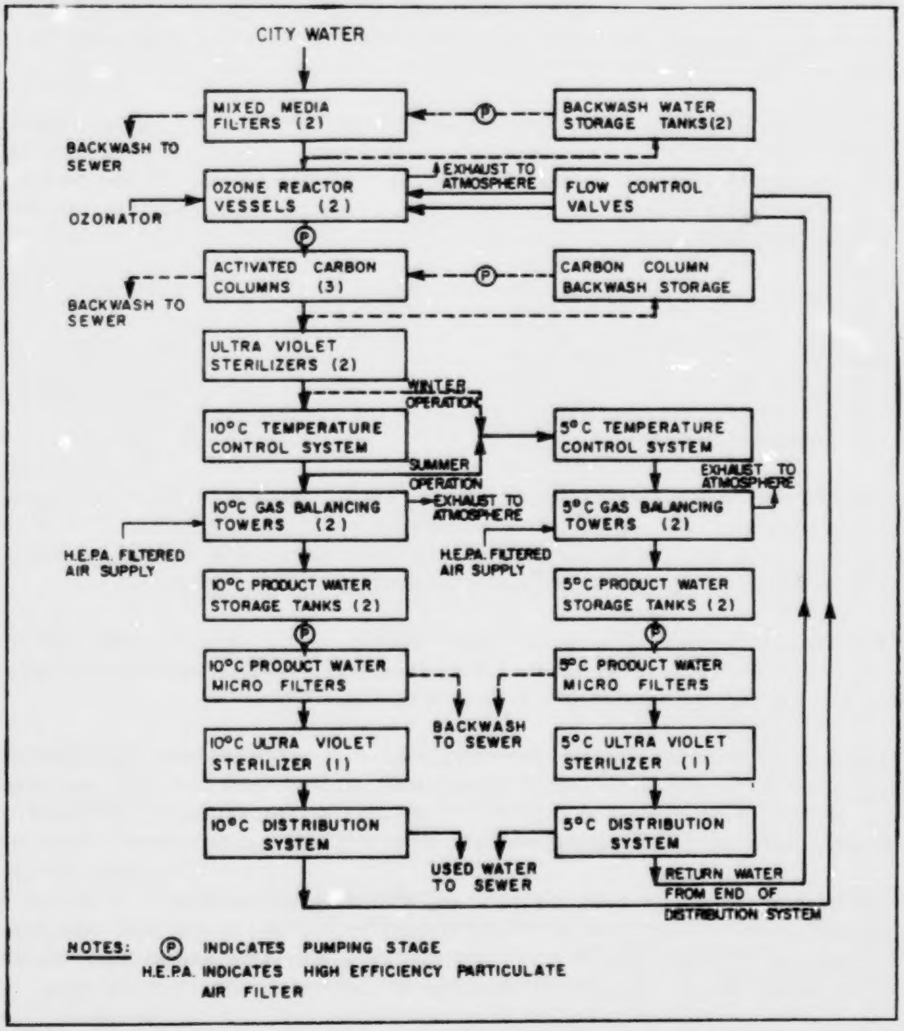
Water temperature control is provided in two stages in a cascade type control system for both 10°C and 5°C streams as follows:

- In the first stage, the water leaving the heating and cooling heat exchangers is controlled to a set point temperature.

- In the second stage, the temperatures in the product water distribution systems are sensed and controllers adjust the set points of the first stage to achieve the desired product water temperatures.

FIGURE 2

Block Diagram of New High Quality Water Treatment System
Freshwater Institute, Winnipeg, Manitoba


CITY WATER
MIXED MEDIA FILTERS (2)
BACKWASH WATER STORAGE TANKS(2)
BACKWASH TO SEWER
OZONE REACTOR VESSELS (2)
EXHAUST TO ATMOSPHERE
FLOW CONTROL VALVES
OZONATOR
ACTIVATED CARBON COLUMNS (3)
CARBON COLUMN BACKWASH STORAGE
BACKWASH TO SEWER
ULTRA VIOLET STERILIZERS (2)
WINTER OPERATION
10°C TEMPERATURE CONTROL SYSTEM
5°C TEMPERATURE CONTROL SYSTEM
SUMMER OPERATION
EXHAUST TO ATMOSPHERE
10°C GAS BALANCING TOWERS (2)
EXHAUST TO ATMOSPHERE
5°C GAS BALANCING TOWERS (2)
H.E.P.A. FILTERED AIR SUPPLY
H.E.P.A. FILTERED AIR SUPPLY
10°C PRODUCT WATER STORAGE TANKS (2)
5°C PRODUCT WATER STORAGE TANKS (2)
10°C PRODUCT WATER MICRO FILTERS
5°C PRODUCT WATER MICRO FILTERS
BACKWASH TO SEWER
10°C ULTRA VIOLET STERILIZER (1)
5°C ULTRA VIOLET STERILIZER (1)
10°C DISTRIBUTION SYSTEM
5°C DISTRIBUTION SYSTEM
USED WATER TO SEWER
RETURN WATER FROM END OF DISTRIBUTION SYSTEM
P

NOTES: Ⓟ INDICATES PUMPING STAGE
H.E.P.A. INDICATES HIGH EFFICIENCY PARTICULATE AIR FILTER

TABLE 2

Design Parameters for Mixed Media Pressure Filters

Item	Design Parameters
Applied Flowrate	640 L/min
Bed Diameter	1.83 m
Bed Area	2.63 m^2
Bed Depth	0.75 m
Bed Composition	MF-186*
Nominal Filtration Rate	0.244 m/min
Nominal Backwashing Rate	0.503 m/min
Nominal Backwashing Flow	1325 L/min
Maximum Expanded Bed Depth	1.5 m
Backwashing Time	10 min
Backwashing Water Quantity	13,260 L
Surface Wash	432 L/min
Nominal Rinse Rate	0.144 m/min
No. of Filters With Above Specs	2

Filtration Equipment Supplier – Pumps & Softeners Ltd.,
Model AMF-72 Duplex

Filter Media Supplier – Neptune-Microfloc Inc.

* Neptune MF-186 Mixed Media: 55% coal – S.G. = 1.6±
30% silica sand – S.G. = 2.6±
15% high density sand – S.G. = 4.0±

TABLE 3

Design Parameters for Ozone Reactor Vessels

Item	Design Parameters
Applied Flowrate	624 L/min
O_3 Demand at 5 mg/L	5.3 kg/day
Ozone/Air Flowrate at 1% O_3 conc	0.28 m^3/min
Nominal Retention Time	5 min
Operating Volume of each Vessel	3000 L
Materials of Construction	Stainless Steel

Ozone supply from two W.R. Grace & Co. Model LG-16 ozone generators installed in the Freshwater Institute's Wastewater Treatment Plant.

TABLE 4

Design Parameters for Activated Carbon Columns

Item	Design Parameters
Applied Flowrate	744 L/min
Bed Diameter	2.13 m
Bed Area	3.56 m^2
Bed Depth	1.5 m
Nominal Filtration Rate	0.209 m/min
Nominal Flow Density	139 L/min/m^3
Superficial Retention Time	7.5 min
Nominal Backwashing Rate	0.391 m/min
Nominal Backwash Flow	1400 L/min
Maximum Expanded Bed Depth	2.25 m
Backwashing Time	10 min
Backwash Water Quantity	14,000 L
Quantity of Activated Carbon	2700 kg
No. of Columns with above specs	3

Activated Carbon Column Equipment Supplier - Pumps & Softeners Ltd.
Model AAC-84 Triplex

Activated Carbon Media Supplier - Witco 59

TABLE 5

Design Parameters for Ultraviolet Sterilizers

Primary Sterilization: (Total Flow Stream)	Two Ultraviolet Purification Systems Units Model EP-10L - 10 Lamp Sterilizers Retention Time = 5.5 sec each at 985 L/min
Secondary Sterilization: (10°C Flow Stream)	One Ultraviolet Purification System Unit Model EP-10L - 10 Lamp Sterilizer Retention Time = 5.5 sec at 985 L/min
Secondary Sterilization: (5°C Flow Stream)	One Ultraviolet Purification System Unit Model EP-8L - 8 Lamp Sterilizer Retention Time = 7.5 sec at 522 L/min
Radiation Intensity:	25,000 microWatt sec/cm^2

TABLE 6
Design Parameters for Gas Balancing Columns

Item	Metric Units
Maximum Applied Flowrate	664 L/min – 10°C System 362 L/min – 5°C System
Tower Diameter	1.8 m 10°C System 1.5 m 5°C System
Packing Depth	2.1 m
Operating Pressure Across Tower	5 cm H_2O
Packing	No. 2 Scrubber-Pak
Number of Columns with above Specs	2 for 10°C System 2 for 5°C System

Vertical Trickle-Flow Packed Column Design
Packing Supplier – Watson Process Systems, Unionville Ontario

TABLE 7
Design Parameters for Product Water Micro-Filters

10°C System:	Bank of 4 cartridge filters with 5-10 micron nylon fabric screen sleeve with dimensions 9 cm diameter x 94 cm length
5°C System:	Bank of 3 cartridge filters with same screen
Backwashing:	Backwashing of individual cartridges initiated automatically at a pre-determined inlet/outlet pressure differential
Manufacturer:	Ronnigen – Petter

TABLE 8
Sizings of Product Water Storage Tanks

Item	Working Capacities
Mixed Media Backwash Tank	12,800 L
Mixed Media Backwash Tank	12,800 L
Act. C Backwash Water Storage Tank	26,000 L
10°C Product Water Tank	10,000 L
10°C Product Water Tank	10,000 L
5°C Product Storage Tank	5,000 L
5°C Product Storage Tank	5,000 L

TREATMENT SYSTEM PERFORMANCE

The second column of numbers in Table 1 indicates the anticipated product water quality for the treatment system. These projections were made during the design phase of the project based upon a review of City of Winnipeg information, limited pilot study data developed by scientists at the Institute, and experience gained at other water treatment facilities. Actual performance data of the treatment plant during the first year of operation (September 1985 to September 1986) have been compiled and reviewed by Freshwater Institute scientists (Wagemann et al 1987a and 1987b). During this period, the flowrate through the treatment system was about half of design capacity. Excerpts of performance data covering the last six months of the first year's operation are presented in the last two columns of Table 1. Comments on the performance of the treatment system, including some extracted from the scientists' report (Wagemann et al 1987a), are as follows:

- **Regarding Chlorine** - After the first six months of operation, the system began to meet consistently the performance criteria for chlorine. The chlorine concentration was for most of the time at or below the target value of 3 ug/L, and was consistently below the anticipated value of 15 ug/L. During the first six months of operation, when the system was still equilibrating, the target value was frequently exceeded but the anticipated value was seldom exceeded. Typical residual chlorine concentrations measured at various points in the treatment process train are as follows:

  City tap water (plant feed) - 350 ug/L
  Activated carbon column fee water - 100 ug/L
  Activated carbon column product water - 20 to 25 ug/L
  Final product water - 0 to 5 ug/L

- **Regarding Ammonia** - The ammonia concentration was consistently higher in the final product water than in untreated City water. Some ammonia was obviously being produced within the system, very likely as a result of bacterial growth, possibly within the activated carbon columns. Although the columns were being backwashed once every week, ammonia production within the system persisted. However, the concentration of ammonia in the product water was always less than the target value of 5 ug/L during the year.

- **Regarding Nitrite and Nitrate** - As with ammonia, nitrite and nitrate concentrations were generally higher in the product water than in City water. The nitrite concentration was consistently below the target value of 20 ug/L but exceeded frequently the anticipated value of 1 ug/L throughout the year. The nitrate concentration consistently exceeded the target value of 10 ug/L, and the anticipated value of 1 ug/L was achieved on only three occasions during the year. From a toxicological point of view, this does not pose a problem because nitrate is not known to be toxic to most organisms even at the highest concentrations (up to 100 ug/L) that were encountered in the product water during the year.

- **Regarding Aluminum** - There was no consistent reduction of dissolved aluminum achieved by the water treatment system. The target value of 1 ug/L in the final product water was exceeded throughout the year, except once, and the anticipated value of 5 ug/L was exceeded about half the time.

- **Regarding Copper** - The copper concentration in the City water was generally at or slightly below the target value of 1 ug/L. Some reduction in the concentration of copper was achieved consistently by the system.

- **Regarding Iron** - The iron content in the City water is due to the iron originally present in Shoal Lake water plus any iron contribution added during the pumping, conveyance, storage and distribution of the City's water supply. Despite some reduction by the treatment system, the iron concentration in the final product water was consistently higher than the target value of 1 ug/L, and also was higher than the anticipated value of 10 ug/L.

- **Regarding Manganese** - The water treatment system was effective in reducing the manganese concentration in the City water, although not consistently. The dissolved manganese concentration in the final product water was significantly lower than in City water - it was frequently at and occasionally below the anticipated value of 2 ug/L. No target value was promulgated for manganese.

- **Regarding Zinc** - The zinc concentration was already quite low in City water and this was further reduced somewhat by the treatment system. The zinc concentration in the final product water therefore never exceeded the target value of 1 ug/L.

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LANDFILL DESIGN IN AREAS OF HIGH WATER TABLES

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INTRODUCTION

Sanitary landfills provide a straight-forward and economical means of disposal of solid wastes and, as a result, are the most common means of final disposal in Western Canada. Most design regulations require that landfills be located a minimum distance above the water table in order to help prevent leachate from entering the groundwater at an unacceptable rate and/or strength. However, in many areas it is difficult to locate the landfill base a sufficient distance above the water table. This paper reviews the current design standards for landfills in Western Canada and presents an innovative alternative for the design of landfills in areas of high water tables.

In this alternative design, the base of the landfill is intentionally placed below the water table, a change in design which has two distinct advantages. The first advantage is that excavating below the water table provides valuable cover material. The second advantage is removal of any free water within the landfill by pumping creases an inward flow of water towards the landfill which, in turn, controls the outward migration of leachate. The rate of the required pumping is minimized by the construction of a liner over the base and sidewalls of the excavation. The resulting design can be an economical and safe solution for the siting and construction of landfills in areas of high water tables.

In most, if not all, solid waste management systems in Western Canada, the means of final disposal is one form or another of sanitary landfill. This is true because sanitary landfills are a relatively "low tech" and affordable disposal option when compared to other more "high tech" and expensive solutions such as incineration. However, sanitary landfills do have problems which must be addressed during planning, design and operation.

Sanitary landfills are sources of potential environmental pollution. The most prevalent manifestations of this pollution potential are air pollution related problems, e.g. dust, smoke, odours; land pollution problems, e.g. wind transported litter; and water pollution problems, e.g. ground water pollution by the leakage of leachate from the sanitary landfill. Other potential problems include various crop damage and/or explosion problems caused by the production and migration of methane gas from the wastes within the landfill.

Sanitary landfills are only an economical disposal solution when the land upon which they are constructed is relatively inexpensive and readily available. It is commonly, but not exclusively, true that such land is of low agricultural value, e.g. the land is stoney or sandy. Unfortunately, this may also mean that there will be more environmental risk associated with these sites and the "special" means of controlling potential pollution must be included in the design of the landfill.

Sanitary landfills must have easily and cheaply available cover material if operation costs are to be kept to a minimum. If cover material must be imported, operating costs will increase dramatically.

As a result of these potential problems, siting of an environmentally safe, aesthetically acceptable and economically cheap landfill becomes a difficult task. This paper addresses this siting problem by illustrating how a relatively new landfill design technique can be used to (i) utilize land that might otherwise be unsuitable for a landfill site, (ii) provide the necessary source of cheap cover material and (iii) provide a means of controlling off-site migration of leachate and pollution of local groundwaters.

CURRENT LANDFILL DESIGN PRACTICE

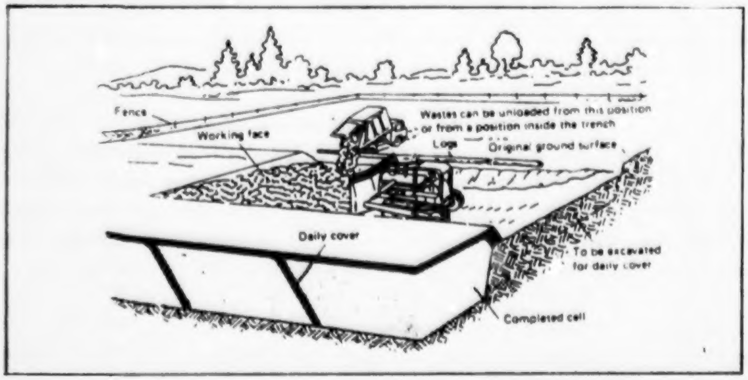
Landfill Design

The most common types of landfill design employed in Western Canada are the trench method and the area method. Both methods have advantages and disadvantages.

The trench method, as the name implies, requires that trenches be excavated into the ground at the site, as shown in Figure 1. Depths are usually in the 3 m (10 ft) range and widths are greater than 4.5 m (15 ft). Lengths are calculated such that the trench is in operation for the fixed period of time, e.g. 3 months, 6 months, etc., not exceeding one year before it is closed and new trench put into use. The main advantage of the trench method is the soil excavated to create the trench can be stockpiled and used as cover material during the operation of the landfill. In the context of this discussion, the main disadvantage is that the site must have a low water table if adequately deep trenches are to be excavated and still fulfill the usual requirement of a minimum distance above the water table. An additional disadvantage is that care must be taken and berms constructed to prevent surface runoff from entering the trench, saturating the wastes and, thereby, creating significant quantities of leachate.

FIGURE 1

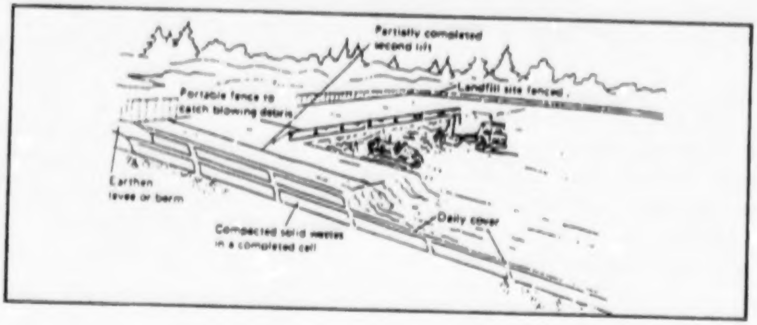
The Trench Method of Landfilling


Fence
Working face
Wastes can be unloaded from this position or from a position inside the trench
Logs
Original ground surface
Daily cover
To be excavated for daily cover
Completed cell

The area method, as shown in Figure 2, is used when cover material is plentiful or when the groundwater level is considered too high for use of the trench method. The area method is relatively simple to operate in that the wastes are simply discharged onto existing ground surface and then worked and compacted into cells. Because of the relatively small differences in elevations, there is less concern with runoff entering the wastes. The main disadvantage of the area method is that the method itself creases very little, if any, cover material and unless there is an abundant and cheap alternate supply of cover material available, cover material costs may be restrictive.

FIGURE 2

The Area Method of Landfilling


Partially completed second lift
Landfill site fenced
Portable fence to catch blowing debris
Earthen levee or berm
Daily cover
Compacted solid wastes in a completed cell

Siting Requirements

The siting, design and operation of sanitary landfills are regulated by the particular provincial departments charged with the task of environmental protection. Typically, the proposed landfill site and design is submitted to an "approvals" branch of these departments for review and recommendations. The "approvals" staff usually, but not always, have regulations or guidelines available to them for use during this review process. In any event, it is often possible to have the proposed site and design reviewed on a "special case" basis.

The regulations or guidelines that do exist [3,6,7,8,] are primarily concerned with preventing real or perceived problems with air, land and water pollution caused by the landfill. The air and land pollution problems are dealt with by prohibiting burning, requiring litter-fencing and maintaining minimum distances from existing or planned housing.

The water pollution problems are primarily dealt with through water table-related siting guidelines. For example, a typical siting guideline suggested by provincial landfill approval authorities [1,3,6] is that no landfill be constructed where the base of the landfill will be less than 1.5 m from the local groundwater table. While it can be argued that there is a problem with what constitutes a 'groundwater table', e.g. whether the water being used for drinking water, etc., there is no argument with the general intent of the regulation, i.e. water pollution prevention. It is assumed that providing at least 1.5 m between the bottom of the landfill and the water table will permit natural dispersion and attenuation processes to decrease the strength of the leachate that may migrate from the landfill (7).

The problem with the water table distance regulation is that it may do one of two things: (i) change the proposed landfill design from the "trench" method to the "area" method and, therefore, immediately cause problems with obtaining sufficient cover material (which would have come from construction of the trenches) or (ii) eliminate that site from further consideration. In both cases, it is likely that the new design or site will be more costly.

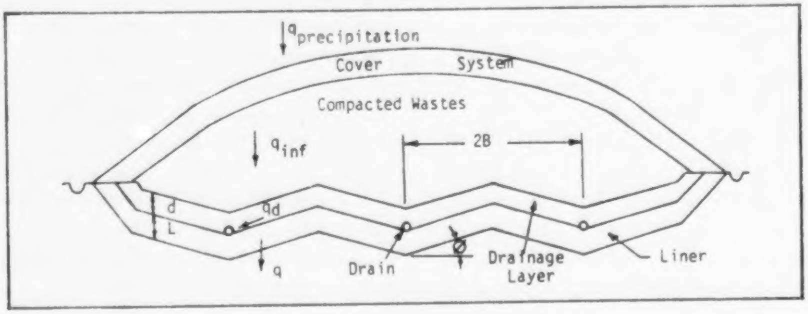
iii) Hydraulic Containment of Leachate

Leachate is essentially a strong wastewater which contains high concentrations of organic and inorganic compounds that have been leached from the wastes in the landfill by liquids, primarily water, which seep through the landfill as a result of (i) being disposed of with the solid wastes, (ii) decomposition of the wastes or (iii) precipitation (rain, snow, etc.). Containment and treatment of this leachate is necessary to prevent potential groundwater pollution. Although specific requirements for leachate containment structures are not usually found in the existing provincial guidelines, they may be required in certain situations, especially when the landfill is being considered under the "special case" situation.

The most common method of leachate containment is lining the bottom of the site with either a synthetic (plastic) membrane or a natural clay-till or bentonite-sand liner (7). As shown in Figure 3, it is necessary to include a leachate collection and removal system whenever a liner is used. The collection system can be used to minimize the hydraulic head across the liner and therefore help to control (but not totally eliminate) leachate leakage. Spacing and sizing of the collection system can be optimised in relationship to the liner design, drainage layer design, predicted leachate production rate(s) and the desired or allowable leachate leakage rates (5).

FIGURE 3

Design Components of a Landfill Leachate Control System



SUMMARY OF EXISTING PRACTICE

From the foregoing, it may be seen that there are two potentially conflicting criteria for successful and economical landfill design and operation. The regulatory authorities desire to prevent water pollution and will, therefore, strive to only approve landfill proposals in which more than the minimum 1.5 m distance to the water table is provided. In addition to this problem, the proponent of the proposed landfill will be concerned about minimizing costs and will be attempting to obtain as much cover from the site as in the trench method. When the water table is high these two criteria come into direct conflict, requiring that either a new site be found or that special design measures be developed.

THE PROPOSED NEW LANDFILL DESIGN

The understanding of the rational of the proposed new landfill design requires discussion of three landfill design scenarios. These scenarios are shown in Figures 4, 5 and 6.

Figure 4 illustrates the situation that is typical of normal landfill approvals on the Prairies: the landfill is located in an area of reasonably "tight" material, i.e. one with a hydraulic conductivity in the 1 x 10^{-6} to 10^{-8} m/s range (1), and the base of the landfill excavation (trench) is more than the minimum 1.5 m above the water table. Typically, under current regulations or guidelines (1,6,7), no liner system is required in this situation. As a result, any leachate that is generated will eventually leak into the water table.

FIGURE 4

A Normal, Non-Lines, Approvable Landfill

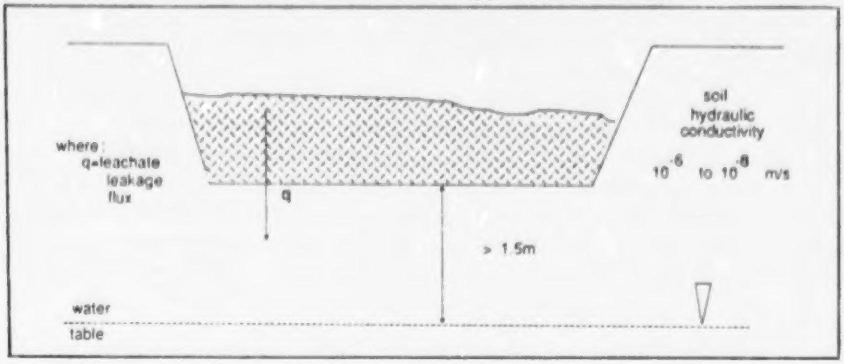
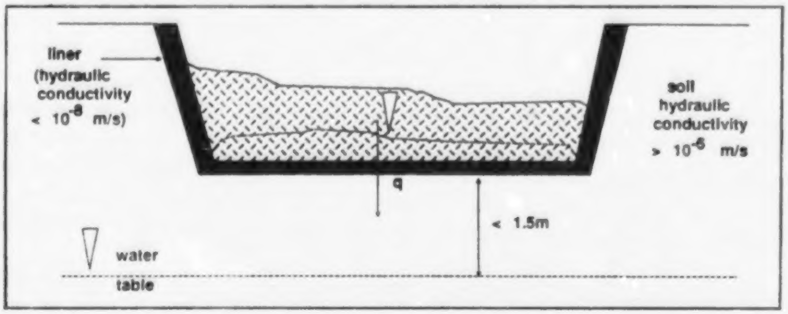

where:
q=leachate leakage flux
q
soil hydraulic conductivity
10^-6 to 10^-8 m/s
> 1.5m
water table

Figure 5 illustrates a more difficult situation in which the ground water table is, for example purposes, less than 1.5 meters from the surface or the natural soil hydraulic conductivity is very high. Under these circumstances, it is likely that a successful landfill design proposal would be one considered as a "special case" and which includes construction of a low permeability clay liner [2,7]. Typically, this liner would be designed to have a hydraulic conductivity of less than 1 x 10^{-9} m/s and a thickness greater than 0.15 m. Such a liner system would decrease, but not totally eliminate, leachate leakage.

Since the solution shown in Figure 5 will help to prevent leachate leakage, but does nothing to help the landfill operator to obtain sufficient quantities of cover material from the site, upward expansion of this landfill may be restricted by the lack of daily cover material. If alternate cover material is very difficult and/or expensive to obtain the site may no longer be economically viable.

FIGURE 5

A Line, Special Case, Approval Landfill

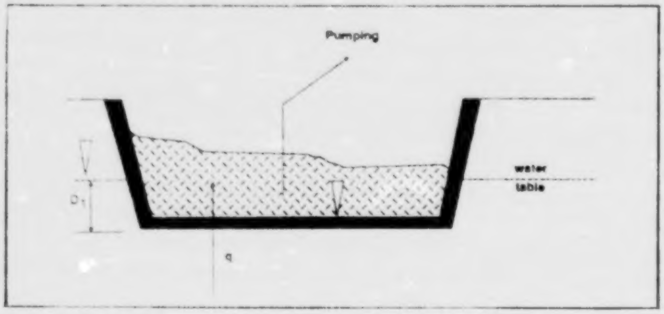

liner
(hydraulic conductivity < 10^-8 m/s)
soil hydraulic conductivity > 10^-6 m/s
q
< 1.5m
water table

This combined high water table-low cover material quantity problem was encountered by the authors during the development of a landfill proposal. As a result, it was necessary to develop an innovative design in which both the criteria of water pollution prevention and provision of cheap[er] cover material were met. The key to this solution was to consider that the water table could be used as a benefit and that, provided the water level within the landfill is kept lower than that of the surrounding water table, the resulting differential head and hydraulic gradient can be used to help contain the leachate within the landfill.

To accomplish this, in the proposed new landfill design, the base of the cell is excavated well below the original groundwater table. In order to keep the leachate-water table gradient favouring flow into rather than out of the landfill, it is necessary to keep the leachate level within the landfill down by pumping it out of the landfill to a holding or treatment pond. Since this could eventually mean that groundwater and not just leachate would be pumped out and that the quantities could get "out of hand", it is necessary to include a liner and collection system as part of this design. The components of this new design concept are shown in Figure 6.

FIGURE 6

The Proposed New Landfill Design



In this new design, the worst case for water flow is when the liner has just been constructed and the full head of the water table, D1, attempts to fill the excavation. However, if the liner is designed and constructed to have, for example, a 0.3 m thickness and 5×10^{-10} m/s hydraulic conductivity, a head of 3 m will only result in an inward seepage of about 0.4 mm of water per day. Since this seepage would only occur during the summer months and typical net evaporation rates are in the 1 to 4 mm/day range, e.g. 3.2 mm/day for Saskatoon, over the May through September period [4], this should not cause any problems. In fact, the seepage would be beneficial by helping to prevent desiccation and cracking problems in the clay-type liners. However, since this moisture may cause some liner swelling and/or vehicle traction problems, it is wise to add a granular layer, e.g. at least 10 cm thick of pit run or graded gravel, above the liner surface to help stabilize any swelling and provide a surface for vehicle traction.

In addition to providing potentially much needed cover material through the deeper than normally permitted excavation, the new below-the-water-table lined landfill (BWTLL) concept also provides for better leachate containment possibilities when compared to the traditional above-the-water-table lined landfill (AWTLL). Furthermore, this is true both during the operation of the site and after closure of the site, regardless of whether or not there is leachate removal by pumping.

During operation of the BWTLL the hydraulic gradient will be very similar to that when that cell was first opened, i.e. there will be a "strong" tendency for flow into but not out of the landfill (except through the leachate collection system). The AWTLL will, however, have at least some flow (leakage) of leachate downwards through the liner as leachate is generated.

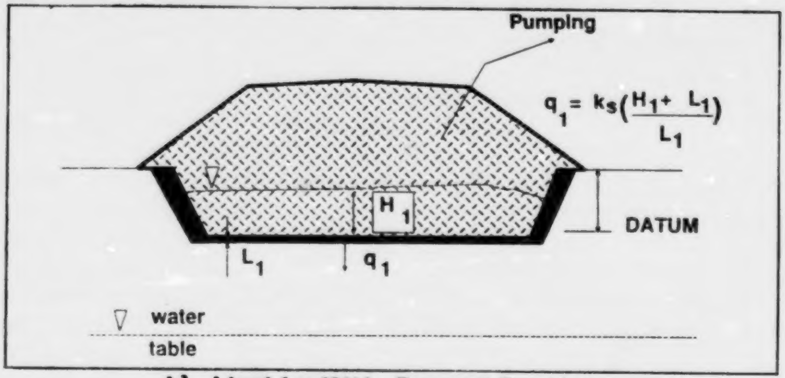
After closure and capping of the landfill, as shown in Figures 7 and 8, the BWTLL is still better than the AWTLL given the same capping and liner conditions. This is true regardless of whether the leachate collection (pumping) system is kept in operation or not.

Figure 7 illustrates the case when the pumping system is kept operational. In the BWTLL case, pumping can be controlled to maintain the interior liquid level, H1, such that it was at or below the exterior water table elevation. As a result, leachate movement across the liner could be kept at zero or negative low, i.e. in-flow. In contract, the AWTLL case there would be a positive head on the liner and, therefore, some leachate leakage downwards through the liner. In fact, the best any AWTLL system can do is only approach zero leachate leakage, albeit at considerable liner thickness and/or collection pipe cost(s).

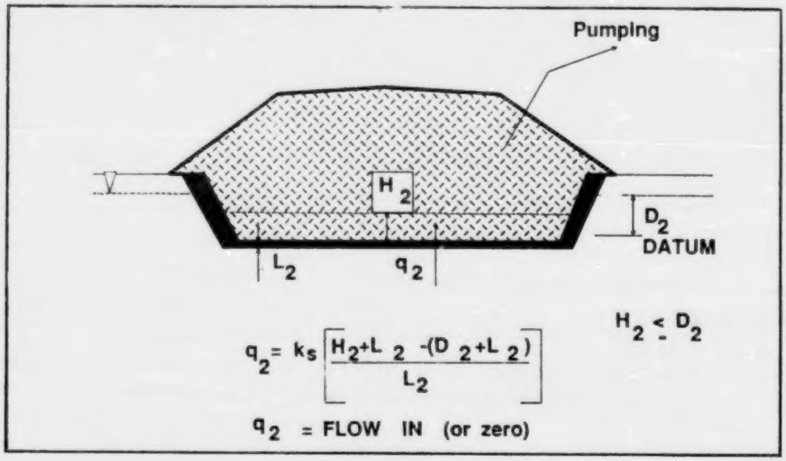
FIGURE 7

Comparison of Current and Proposed Design Practice, With Pumping

a) Lined Landfill, Current Practice



b) Lined Landfill, Proposed Practice

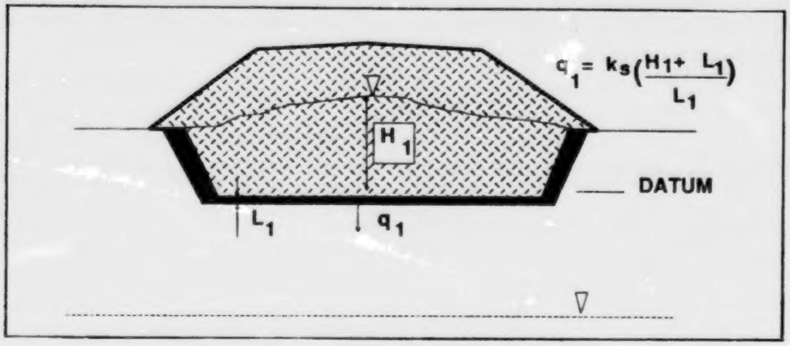

Pumping
H_2
D_2
DATUM
L_2
q_2
$H_2 \leq D_2$
$q_2 = k_s\left[\frac{H_2+L_2-(D_2+L_2)}{L_2}\right]$
q_2 = FLOW IN (or zero)

In the case when the pumping system is not operated after closure, as shown in Figure 8, the BWTLL system is still better than the AWTLL system. For example, if similar capping and liner conditions are assumed, the build up or mounding of leachate within both systems would be similar. Given this assumption, the BWTLL system liner would only have to contend with the head above the exterior water table elevation, whereas the AWTLL system liner is affected by the full mounding height head. As a result, the leachate leakage flux, q, will always be higher in the AWTLL case than in the BWTLL case.

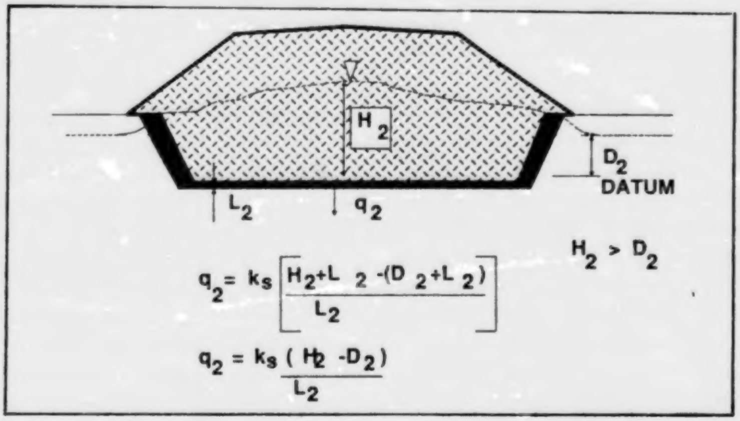
FIGURE 8

Comparison of Current and Proposed Design Practice, With No Pumping

a) Lined Landfill, Current Practice



b) Lined Landfill, Proposed Practice


H_2
D_2
DATUM
L_2
q_2
$H_2 > D_2$
$q_2 = k_s\left[\frac{H_2 + L_2 - (D_2 + L_2)}{L_2}\right]$
$q_2 = k_s\frac{(H_2 - D_2)}{L_2}$

CONCLUSIONS

The proposed new below-the-water table lined landfill (BWTLL) design offers several advantages over the typical above-the-water table lined landfill (AWTLL) design. the BWTLL concept (i) permits use of relatively wet land which otherwise would not be considered for a landfill site, (ii) provides a source of cheap, on-site cover material and (iii) provides a method of leachate containment that can, potentially, totally eliminate leachate leakage. Disadvantages of the

BWTLL design include requirements for (i) site dewatering during construction of the liner system and (ii) the potential need for leachate collection pumping during site operation and closure.

Further research in this area could deal with modelling the system to include both hydraulic and chemical gradients so that the requirements for hazardous waste containment landfills can be better determined.

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SAFETY - WHAT'S IN IT FOR ME?

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This presentation describes the benefits (absence of pain and suffering, improved productivity, reduced costs, improved labour relations, etc.) of a successful safety program from the perspectives of a senior manager, a middle manager, a laborer and a union representative.

While this presentation will be of interest to all, it is aimed at managers - those individuals responsible for the development and implementation of safety programs that will safeguard their most valuable resource - people.

* Additional information may be obtained directly from the author.

EXPERIENCES IN PROVIDING HIGH LEVEL WASTEWATER TREATMENT AT AN ONTARIO MILK PRODUCING FACILITY

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The Winchester plant of Ault Foods Limited, south of Ottawa, is the largest butter, cheese and milk powder producer in Ontario. From its beginning in 1891 it has discharged wastewater to a small creek. Over time a treatment system consisting of aerated cells and holding lagoons was developed. An innovative demineralized whey powder process installed in 1985 accentuated already severe discharge problems.

This paper traces the problems encountered in securing provincial discharge approval under Ontario's new strict regulations. A phased programme of facility improvements is in effect, including activated sludge treatment, phosphorus removal, and sludge management.

* Additional information may be obtained directly from the author.

ORGANIC CONTAMINANTS IN CANADIAN MUNICPAL SLUDGES

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Organic contaminant (priority pollutant) analysis was conducted for a cross-section of Canadian municipal sludges. Several of the sludges were derived from large cities where there is concentration of industry and a recognized potential for contamination of wastewater streams. In general, organic contaminant concentrations in the sludges were low with many reports of no detection or a trace amount. Of 77 contaminants tested, 15 were not detected in any of the sludges and 62 were detected in one or more sludges. Twenty-five occurred at concentration <1, 40 occurred at <10 and 54 occurred at <100 mg kg^{-1} dw. Eight PAH and phthallates exceeded 100 mg kg^{-1}. The City of Hamilton is heavily industrialized with coking and foundry operations and all PAH concentrations exceeding 10 mg kg^{-1} occurred in the Hamilton sludge. The implications of these results for sludge use in agriculture will be discussed.

* Additional information may be obtained directly from the author.